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Seismic Vulnerability Assessment of Selected Facilities at Three SAC Air Force Bases in California

by
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This report summarizes the seismic vulnerability assessment of selected mission essential and high-potential-loss facilities at three Strategic Air Command (SAC) bases in California: Beale, Castle, and March Air Force Bases.

Buildings that may be severely damaged during a major earthquake are identified, preliminary seismic analyses performed, and concepts and preliminary cost estimates for upgrading the facilities are developed.

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Buildings that may be severely damaged during a major earthquake are identified, preliminary seismic analyses performed, and concepts and preliminary cost estimates for upgrading the facilities are developed.

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FOREWORD

This study was performed for U.S. Air Force, Headquarters, Strategic Air Command (HQ SAC), Directorate of Engineering and Services, by the Engineering and Materials Division (EM), U.S. Army Construction Engineering Research Laboratory (USACERL) under Purchase Order WES-PO-02, dated September 1989 from the U.S. Army Engineer Waterways Experiment Station (USAEWES). The HQ SAC technical monitor was Mr. Scott Newquist, DEMM. The USAEWES technical monitor was Dr. Robert Hall, CEWES-SS-R.

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COL Everett R. Thomas is Commander and Director of USACERL and Dr. L.R. Shaffer is Technical Director.

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1 INTRODUCTION

Background

Building design to resist earthquake loads seeks to ensure life safety. Present building codes are based on the philosophy that structures should resist minor earthquakes without structural damage, moderate earthquakes with some structural damage, and major earthquakes without collapse. In addition, essential facilities at military installations must maintain military readiness and support surrounding communities in vital postearthquake relief and rescue operations. As more data on earthquake effects is obtained, it has become necessary to increase the design loads of older codes and change some detailing requirements to improve the seismic resistance of buildings. While new construction has generally incorporated these provisions, many existing military facilities designed in accordance with provisions of earlier (pre-1970) prevailing building codes remain vulnerable.

The West Coast of the United States is a region of high seismicity. Therefore, it is extremely important to perform vulnerability assessments of existing military structures in this region to identify structural deficiencies found in these facilities. Three Strategic Air Command (SAC) bases are located in California: Castle Air Force Base (AFB), Beale AFB, and March AFB.

Headquarters, Strategic Air Command (HQ SAC) initiated a program to assess the seismic vulnerabilities at these bases so that it can develop structural upgrades to those facilities requiring strengthening. HQ SAC asked the U.S. Army Construction Engineering Research Laboratory (USACERL) and the U.S. Army Engineer Waterways Experiment Station (USAEWES) to perform this work.

Castle and Beale AFBs are in Seismic Zone 3 and March AFB is in Seismic Zone 4 as defined by the Uniform Building Code. Selected facilities at these bases were analyzed and the results of the seismic vulnerability assessments are summarized in this report.

Objective

The objective of this work was to determine the earthquake vulnerabilities of selected essential and high-potential-loss facilities at Beale, Castle, and March AFBs to ensure life safety and minimize potential mission disruption.

Approach

The first step in this evaluation was to conduct an inventory reduction on the large number of facilities at the three bases. Buildings selected for evaluation are representative of a group of buildings with similar construction. Preliminary seismic safety evaluations were then conducted. Upgrading

concepts and preliminary cost estimates for upgrading were developed. This evaluation was performed as prescribed by the triservice manuals.[1]

Scope

The information in this report is limited to approximate analysis of earthquake ground shaking effects on buildings. This preliminary analysis provides the information needed to identify facilities with serious damage potential. For those facilities, detailed analyses, which are beyond the scope of this project, must be performed and construction drawings must be developed for complete cost estimates of upgrading.

[1] Technical Manual (TM) 5-809-10-1, Navy Manual NAVFAC P-355.1, Air Force Manual (AFM) 88-3, Chap 13, Sec A; *Seismic Design Guidelines for Essential Buildings* (Departments of the Army, the Navy, and the Air Force, February 1986); TM 5-809-10-2, NAVFAC P-355.2, AFM 88-3, Chap 13, Sec B, *Seismic Design Guidelines for Upgrading Existing Buildings* (Departments of the Army, the Navy, and the Air Force, September 1988).

2 SEISMIC LOAD LEVEL

Site-specific Response Spectra

AFM 88-3, Chap 13, Sec A specifies two risk levels of seismic ground motion for evaluating existing facilities. These risk levels are a 50 percent risk of exceedance in 50 years (EQ I), and a 10 percent risk of exceedance in 100 years (EQ II). Effective peak ground accelerations were developed by Dr. Ellis Krinitzsky, USAEWES, for Beale, Castle, and March AFBs in accordance with the recommended probabilistic method outlined in Chapter 3 of AFM 88-3, Chap 13, Sec A and these two risk levels. These values are shown in Table 1. Documentation of the development of the effective peak acceleration is provided in a USAEWES report.[2]

Design Response Spectra

USACERL developed design response spectra based on these effective peak ground accelerations. A design response spectra is defined as the graphical representation of the maximum response of a single-degree-of-freedom elastic system with damping to a given dynamic motion or force. The abscissa of the

Table 1

Effective Peak Accelerations

Site*	Probability of Exceedance (Percent)	Acceleration for Exposure Time, g 50 Years	100 Years
Beale			
	10	NA	.291
	50	.204	NA
Castle			
	10	NA	.275
	50	.194	NA
March			
	10	NA	.505
	50	.327	NA

*Soft site, near field source, mean.

[2] D.W. Sykora, et al., *Earthquake Investigations for SAC Bases in California: Beale, Castle, and March AFBs - Design Accelerations and Response Spectra*, Draft Report (U.S. Army Engineer Waterways Experiment Station [USAEWES], August 1990).

spectrum is the natural frequency or natural period of the system and the ordinate is the maximum response. The spectra were developed using a spectral construction method recommended by Newmark and Hall.[3] To construct the design spectra, the anticipated effective peak acceleration, velocity, and displacement are required. Velocity and displacement parameters were determined using the following ratios:

$$v/a = 48 \text{ in./sec/g}^{*}$$

where v = maximum effective horizontal velocity (in./sec)

a = maximum effective horizontal acceleration (gravity [g]).

and

$$ad/v^2 = 6.0$$

where d = maximum effective horizontal displacement (in.).

These values are given in Newmark and Hall.[4] The values for v/a and ad/v^2 agree with similar values given by Seed and Idriss[5] and Mohraz, Hall, and Newmark[6] for soft soil conditions.

The design spectra for varying levels of critical damping were then constructed by multiplying the ground motion parameters by the 84 percent spectrum amplification values from Newmark and Hall.[7] The values are reproduced in Table 2. The calculated design spectra values for EQ-I are shown in Tables 3, 5, and 7 for Beale, Castle, and March AFBs, respectively, and for EQ-II in Tables 4, 6, and 8. The curves can then be plotted directly on tripartite graph paper. The design response spectra are shown graphically in Figures 1 through 3.

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[3] N.M. Newmark and W.J. Hall, "Earthquake Spectra and Design," *Earthquake Engineering Research Institute Monograph Series* (1982).

[*] A metric conversion table is provided on p 67.

[4] N.M. Newmark and W.J. Hall, p 45.

[5] H.B. Seed and I.M. Idriss, "Ground Motions and Soil Liquefaction During Earthquakes," *Earthquake Engineering Research Institute Monograph Series.*

[6] B. Morhaz, W.J. Hall, and N.M. Newmark, *A Study of Vertical and Horizontal Earthquake Spectra* (Division of Reactor Standards, U.S. Atomic Energy Commission, December 1972).

[7] N.M. Newmark and W.J. Hall, p 35.

Table 2

Spectrum Amplification Factors For Horizontal Elastic Response

% Critical Damping	One Sigma (84.1%) A	V	D	Median (50%) A	V	D
0.5	5.10	3.84	3.04	3.68	2.59	2.01
1	4.38	3.38	2.73	3.21	2.31	1.82
2	3.66	2.92	2.42	2.74	2.03	1.63
3	3.24	2.64	2.24	2.46	1.86	1.52
5	2.71	2.30	2.01	2.12	1.65	1.39
7	2.36	2.08	1.85	1.89	1.51	1.29
10	1.99	1.84	1.69	1.64	1.37	1.20
20	1.26	1.37	1.38	1.17	1.08	1.01

Table 3

Digitized Design Spectra for Beale AFB - EQ I

Period	Percent of Critical Damping 3	5	7	10
0.00	0.66	0.55	0.48	0.40
0.10	0.66	0.55	0.48	0.40
0.20	0.66	0.55	0.48	0.40
0.30	0.66	0.55	0.48	0.40
0.40	0.66	0.55	0.48	0.40
0.50	0.66	0.55	0.48	0.40
0.60	0.66	0.55	0.48	0.40
0.65	0.64	0.55	0.48	0.40
0.70	0.60	0.52	0.47	0.40
0.73	0.57	0.50	0.45	0.40
0.75	0.56	0.48	0.44	0.39
0.80	0.52	0.45	0.41	0.36
0.85	0.49	0.43	0.39	0.34
0.90	0.46	0.40	0.36	0.32
1.00	0.42	0.36	0.33	0.29
1.10	0.38	0.33	0.30	0.26
1.20	0.35	0.30	0.27	0.24
1.30	0.32	0.28	0.25	0.22
1.40	0.30	0.26	0.23	0.20
1.50	0.28	0.24	0.22	0.19
2.00	0.21	0.18	0.16	0.14
2.50	0.16	0.14	0.13	0.11
3.00	0.12	0.11	0.10	0.09

Table 4

Digitized Design Spectra for Beale AFB - EQ II

Period	Percent of Critical Damping			
	3	5	7	10
0.00	0.94	0.78	0.68	0.57
0.10	0.94	0.78	0.68	0.57
0.20	0.94	0.78	0.68	0.57
0.30	0.94	0.78	0.68	0.57
0.40	0.94	0.78	0.68	0.57
0.50	0.94	0.78	0.68	0.57
0.60	0.94	0.78	0.68	0.57
0.65	0.92	0.78	0.68	0.57
0.70	0.85	0.74	0.67	0.57
0.75	0.80	0.69	0.63	0.55
0.80	0.75	0.65	0.59	0.52
0.85	0.70	0.61	0.55	0.49
0.90	0.66	0.58	0.52	0.46
1.00	0.60	0.52	0.47	0.41
1.10	0.54	0.47	0.42	0.38
1.20	0.50	0.43	0.39	0.34
1.30	0.46	0.40	0.36	0.32
1.40	0.42	0.37	0.33	0.29
1.50	0.40	0.34	0.31	0.27
2.00	0.30	0.26	0.23	0.20
2.50	0.24	0.20	0.18	0.16
3.00	0.17	0.15	0.14	0.13

Table 5

Digitized Design Spectra for Castle AFB - EQ I

Period	Percent of Critical Damping			
	3	5	7	10
0.00	0.63	0.53	0.46	0.39
0.10	0.63	0.53	0.46	0.39
0.20	0.63	0.53	0.46	0.39
0.30	0.63	0.53	0.46	0.39
0.40	0.63	0.53	0.46	0.39
0.50	0.63	0.53	0.46	0.39
0.60	0.63	0.53	0.46	0.39
0.65	0.62	0.53	0.46	0.39
0.70	0.57	0.50	0.45	0.39
0.75	0.53	0.46	0.42	0.37
0.80	0.50	0.44	0.39	0.35
0.85	0.47	0.41	0.37	0.33
0.90	0.44	0.39	0.35	0.31
1.00	0.40	0.35	0.31	0.28
1.10	0.36	0.32	0.29	0.25
1.20	0.33	0.29	0.26	0.23
1.30	0.31	0.27	0.24	0.21
1.40	0.29	0.25	0.22	0.20
1.50	0.27	0.23	0.21	0.19
2.00	0.20	0.17	0.16	0.14
2.50	0.16	0.14	0.13	0.11
3.00	0.12	0.11	0.10	0.09

Table 6

Digitized Design Spectra for Castle AFB - EQ II

Period	Percent of Critical Damping			
	3	5	7	10
0.00	0.89	0.745	0.64	0.54
0.10	0.89	0.745	0.64	0.54
0.20	0.89	0.745	0.64	0.54
0.30	0.89	0.745	0.64	0.54
0.40	0.89	0.745	0.64	0.54
0.50	0.89	0.745	0.64	0.54
0.60	0.89	0.745	0.64	0.54
0.65	0.87	0.745	0.64	0.54
0.70	0.81	0.705	0.63	0.54
0.75	0.75	0.658	0.59	0.52
0.80	0.70	0.617	0.55	0.49
0.85	0.66	0.581	0.52	0.46
0.90	0.63	0.549	0.49	0.43
1.00	0.56	0.494	0.44	0.39
1.10	0.51	0.449	0.40	0.35
1.20	0.47	0.411	0.37	0.32
1.30	0.43	0.380	0.34	0.30
1.40	0.40	0.353	0.31	0.28
1.50	0.37	0.329	0.29	0.26
2.00	0.28	0.247	0.22	0.19
2.50	0.22	0.197	0.17	0.15
3.00	0.16	0.150	0.13	0.12

Table 7

Digitized Design Spectra for March AFB - EQ I

Period	Percent of Critical Damping			
	3	5	7	10
0.00	1.06	0.89	0.77	0.65
0.10	1.06	0.89	0.77	0.65
0.20	1.06	0.89	0.77	0.65
0.30	1.06	0.89	0.77	0.65
0.40	1.06	0.89	0.77	0.65
0.50	1.06	0.89	0.77	0.65
0.60	1.06	0.89	0.77	0.65
0.65	1.04	0.89	0.77	0.65
0.70	0.96	0.84	0.76	0.65
0.75	0.90	0.78	0.71	0.63
0.80	0.84	0.73	0.66	0.59
0.85	0.79	0.69	0.62	0.55
0.90	0.75	0.65	0.59	0.52
1.00	0.67	0.59	0.53	0.47
1.10	0.61	0.53	0.48	0.43
1.20	0.56	0.49	0.44	0.39
1.30	0.52	0.45	0.41	0.36
1.40	0.48	0.42	0.38	0.34
1.50	0.45	0.39	0.35	0.31
2.00	0.34	0.29	0.27	0.23
2.50	0.27	0.23	0.21	0.19
3.00	0.20	0.18	0.16	0.15

Table 8

Digitized Design Spectra for March AFB - EQ II

Period	Percent of Critical Damping			
	3	5	7	10
0.00	1.63	1.36	1.19	1.00
0.10	1.63	1.36	1.19	1.00
0.20	1.63	1.36	1.19	1.00
0.30	1.63	1.36	1.19	1.00
0.40	1.63	1.36	1.19	1.00
0.50	1.63	1.36	1.19	1.00
0.60	1.63	1.36	1.19	1.00
0.65	1.60	1.36	1.19	1.00
0.70	1.48	1.29	1.17	1.00
0.75	1.38	1.20	1.09	0.96
0.80	1.30	1.13	1.02	0.90
0.85	1.22	1.06	0.96	0.85
0.90	1.15	1.00	0.91	0.80
1.00	1.04	0.90	0.82	0.72
1.10	0.94	0.82	0.74	0.65
1.20	0.86	0.75	0.68	0.60
1.30	0.80	0.69	0.63	0.55
1.40	0.74	0.64	0.58	0.51
1.50	0.69	0.60	0.54	0.48
2.00	0.52	0.45	0.41	0.36
2.50	0.41	0.36	0.32	0.29
3.00	0.30	0.27	0.25	0.23

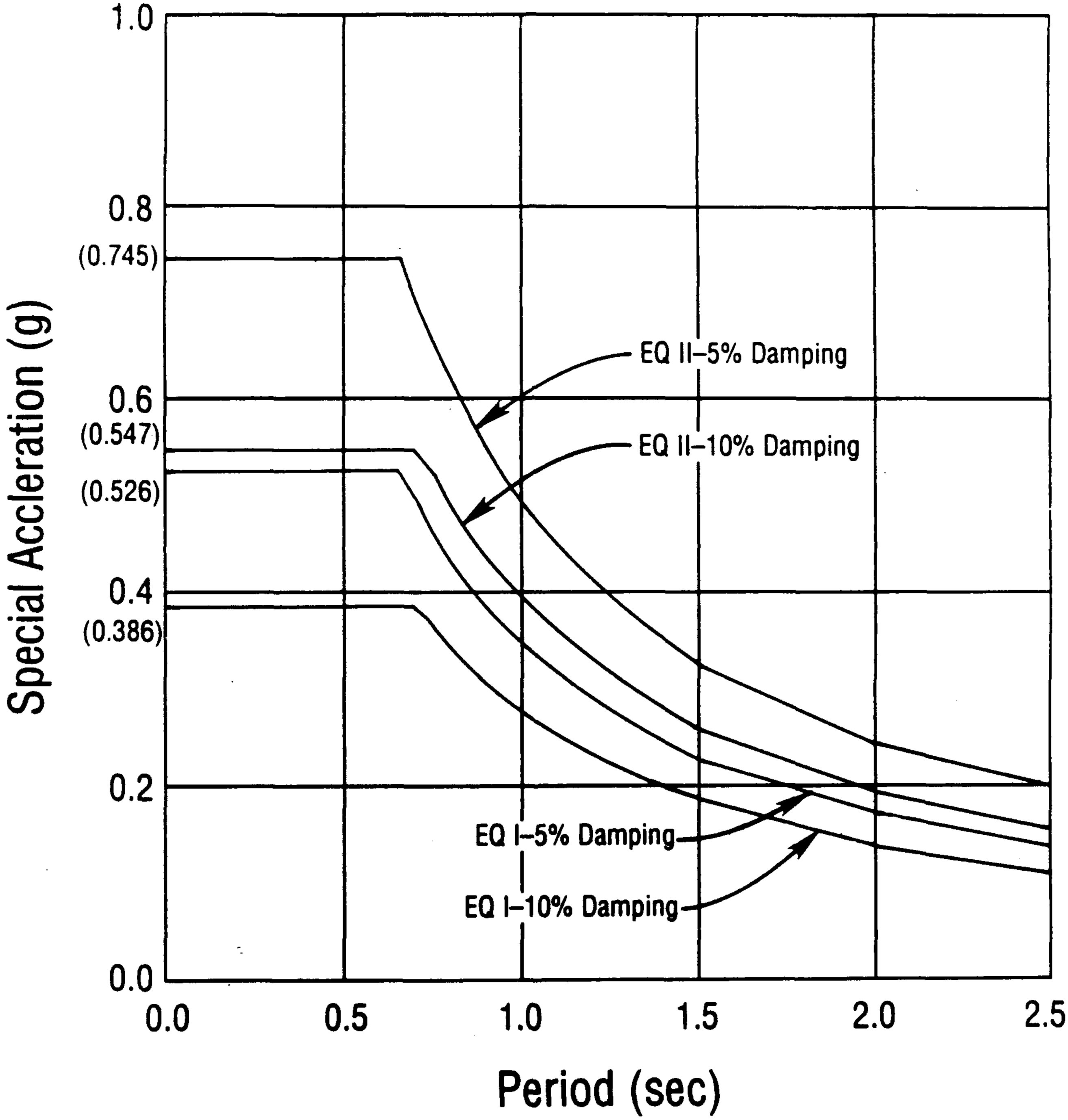

Figure 1. Design response spectra, Beale AFB - EQ I and EQ II.

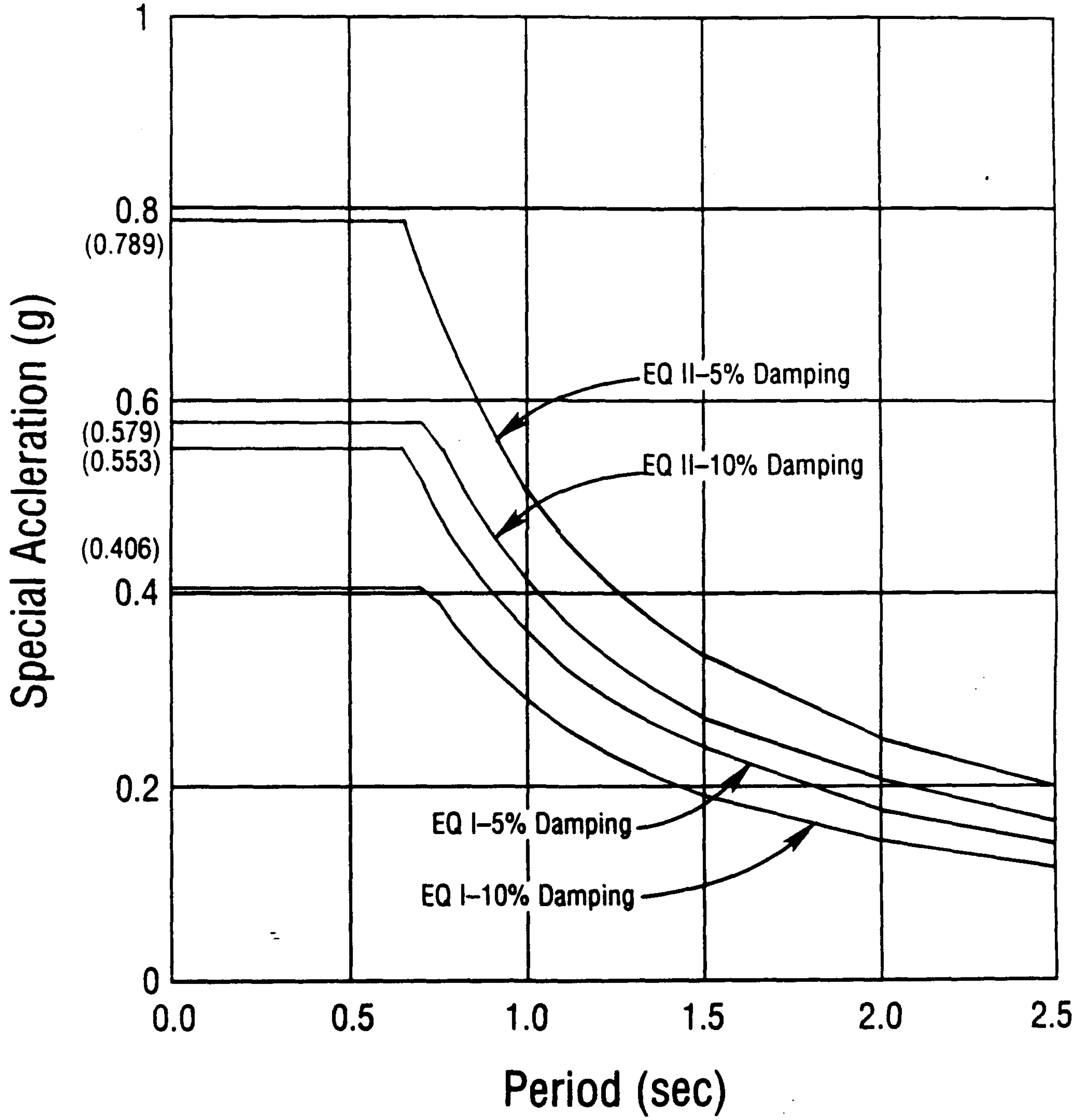

Figure 2. Design response spectra, Castle AFB - EQ I and EQ II.

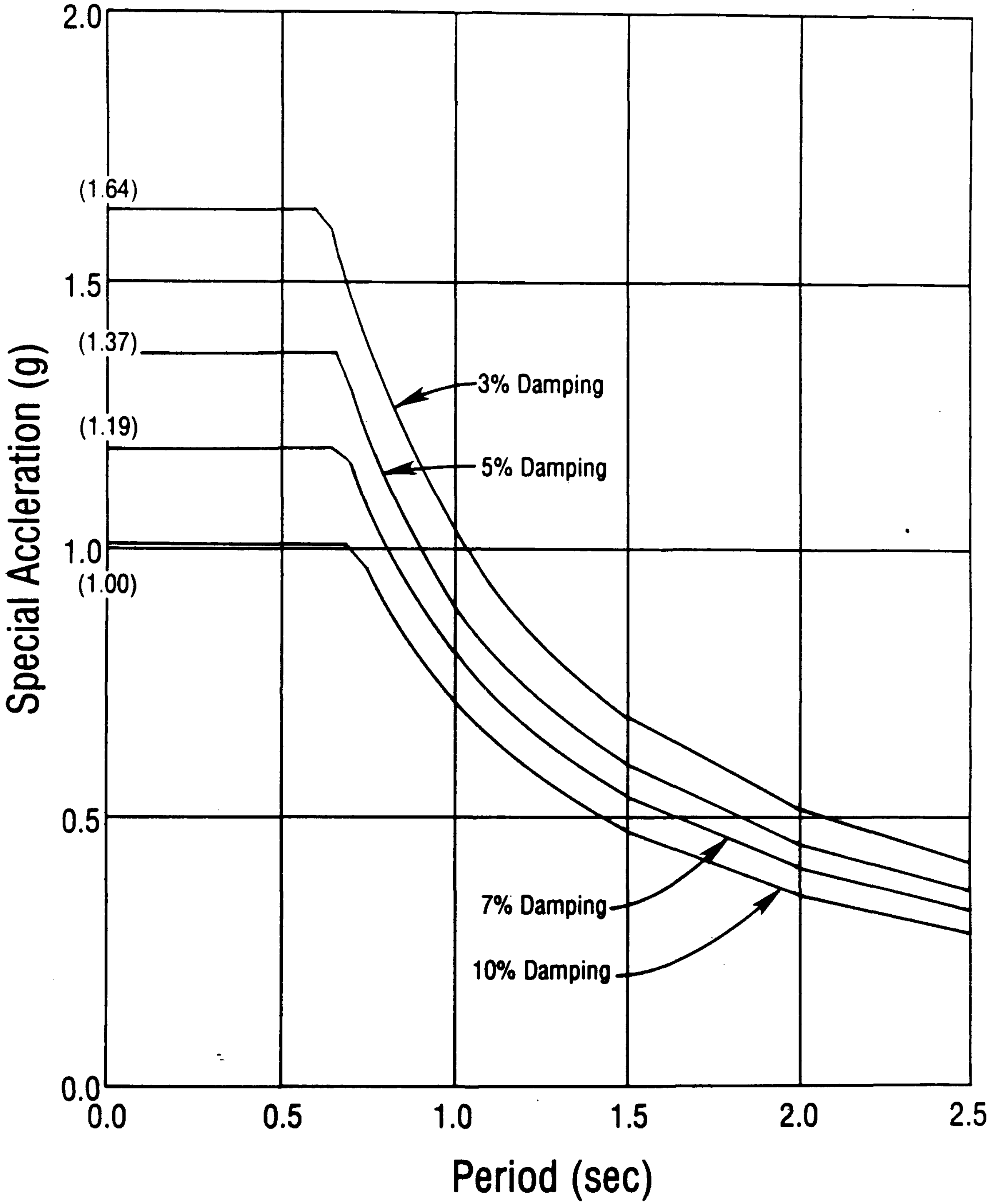

Figure 3. Design response spectra, March AFB - EQ II.

3 SELECTION OF BUILDINGS

Inventory Reduction and Preliminary Screening

The inventory of facilities included in this study was selected by Base Civil Engineer personnel at each of the bases with the concurrence of HQ SAC/DEMM. Base Civil Engineers selected 22 facilities at Castle AFB; 15 facilities at Beale AFB were selected from the list of facilities remaining after the inventory reduction. At March AFB, 39 facilities passed the inventory reduction conducted by the Base Civil Engineer.

USACERL and USAEWES engineers reviewed design data and conducted a field inspection of the selected buildings. The buildings were then screened to eliminate those that were structurally similar (for which a single representative structure could be analyzed). In reviewing the available design documents, some buildings lacked drawings or critical structural data. Because these buildings would have required development of detailed structural data (a task outside the scope of this assessment) they were not analyzed. The facilities that could not be analyzed and the reasons for this are listed in Tables 9, 10, and 11.

Table 9

Buildings Eliminated From Preliminary Analysis at Beale AFB

Building No.	Reason for Elimination
1046	*Structural information is not available for the prefabricated frames. Flange and web thicknesses, stiffener sizes, and selected frame dimensions are missing.*

Table 10

Buildings Eliminated From Preliminary Analysis at Castle AFB

Building No.	Reason for Elimination
54	No design drawings available
480	Similar to Building 1212
1107	Drawings lack critical detailing information
1182	Seismic analysis performed during recent upgrading
1319	Structural information not available for prefabricated steel frames

Table 11

Buildings Eliminated From Preliminary Analysis at March AFB

Building No.	Reason for Elimination
100	No plans available
355	Similar to Building 300
373	Similar to Building 300
385	Similar to Building 300
420	No plans available
429	Similar to Building 300
440	Similar to Building 300
441	No plans available
449	No plans available
452	Similar to Building 300
457	Similar to Building 300
463	Insufficient plans available
758	Insufficient plans available
2307	Missed in the initial survey
2419	No plans are available
2420	Similar to Building 2418
2421	Similar to Building 2420
3408	Similar to Building 3407
3417	Similar to Building 3418

Seventeen buildings remained after preliminary screening at Castle AFB. Fifteen facilities remained after screening at Beale AFB; twenty-one facilities remained after screening at March AFB. The facilities remaining after the preliminary screening are listed Tables 12, 13, and 14 for Beale, Castle, and March AFBs, respectively. A summary description of each building is provided in the following section.

Beale AFB Building Descriptions

Building 1025 - Avionics Maintenance

The avionics maintenance facility consists of two buildings; the original composite maintenance shop and the avionics shop addition. Both are one-story, rectangular buildings. The perimeter of both portions of the facility is constructed of precast concrete panels. The avionics shop is seismically separated from the original structure by a 2 1/2-in. joint. General notes on the structural drawings specify continuity requirements between structural elements for the avionic shop addition. Due to its recent construction and the reasons just stated, the seismic capacity of this building was not analyzed further.

The vertical load carrying system of the composite maintenance shop consists of wide-flange steel beams and open-web steel joists that carry the roof loads to interior and exterior steel columns. Perimeter precase concrete walls are nonload-bearing. Continuous reinforced concrete strip footings support the

Table 12

Facilities at Beale AFB Screened for Seismic Vulnerability Evaluation

Building No.	Building Description
1025	Avionics Maintenance
1029	Physiological Support
1060	Base Operations and Control Tower
1086	Command Post
1200	Air Refueling Squad
2145	Continuous Processing
2340	Officers Club
2418	Gymnasium
2434	Exchange
2459	Commissary
2474	Theater
2490	Dining Hall
5700	Hospital
5800	NCO Club

Table 13

Facilities at Castle AFB Screened for Seismic Vulnerability Evaluation

Building No.	Building Description
175	Weapon System Training
360	Dining Hall
752	Commissary
759	Base Exchange
786	Theater
1212	Law Enforcement Center
1230	93rd Bombardment Wing Command Post
1260	Jet Engine Inter Maintenance
1325	Field Maintenance Squadron
1335	Avionics Maintenance Squadron
1340	Base Operations/Tower
1344	Fire Station
1350	Maintenance Hangar
1360	Base Supply
1532	Precision Measurement Equipment
1540	Squadron Operations
1582	SAC Alert Facility

Table 14

Facilities at March AFB Screened for Seismic Vulnerability Evaluation

Building No.	Building Description
300	Hangar/Warehouse
465	Physical Fitness Center
651	Recreation Center
760	Theater
960	Commissary
962	Dining Facility
1220	Base Operations and Control Tower
1223	Fire Station
1305	Readiness Crew
2300	Base Contracting
2303	Hangar Maintenance
2310	Warehouse Supply
2418	'ormitory
2595	Child Care Center
2605	15th AF Combat Operations
2630	33rd Communications Group Headquarters
2706	NonCommissioned Officers' Club
3403	15th AF Headquarters
3407	Dormitory
3417	Dormitory

perimeter walls and concrete pedestals and square footings support the columns. Lateral loads are transferred by the concrete roof diaphragm to the precast concrete walls. A field inspection showed the facility to be in good condition.

A review of the structural drawings revealed that the precast walls are positively joined to the roof diaphragm and steel columns by a connection of plates, welds, and anchor bolts. The walls are connected to the foundation by reinforcing steel. In addition, the precast panels rest on a grout bed that provides some shear friction resistance to lateral load. However, in this preliminary analysis, the capacity of the shear walls was assumed to be resisted solely by the welded connections.

Building 1029 - Physiological Support/High Altitude Training

The physiological support/high altitude training facility is composed of the original physiological support structure and the high altitude training addition to the west of the original building. The physiological support building is essentially a one-story building with a two-story high portion at one end. Its vertical load carrying system consists of open-web steel joists and wide-flange steel beams that carry the roof loads to cast-in-place concrete pilasters, steel pipe columns, and interior and exterior load-bearing precast concrete walls. Continuous, reinforced concrete strip footings under bearing walls, and spread footings under columns and pilasters form the foundation. The lateral load resisting system consists of a diaphragm of lightweight concrete over metal deck and precast and cast-in-place concrete shear walls in the transverse and longitudinal directions. There is no reinforcing in the joint between the precast walls and the foundation.

The high altitude training building is one-story and essentially rectangular in plan. Vertical loads are supported by open-web steel joists and wide-flange steel beams that carry the roof load to steel pipe columns, and interior and exterior concrete walls. Continuous, reinforced concrete grade beams span to drilled piers beneath cast-in-place concrete wall elements. Drilled concrete piers also support the vertical loads transferred by steel pipe columns. Lateral loads are resisted by precast concrete shear walls in the transverse and longitudinal directions. The joint between these walls and the foundation contains no reinforcement. Drilled piers resist overturning of the lateral system. Both buildings appeared to be in good condition when inspected.

In the lateral analysis, researchers assumed that the two portions of the building act as one system in resisting lateral loads. As there is no reinforcing in the joints between the precast walls and foundation, researchers assumed shear was resisted by friction of the dry pack grout against the hardened concrete surfaces. The capacity of the concrete pilasters to resist seismic loads was not evaluated.

Building 1069 - Base Operations and Control Tower

Base operations is a one-story building rectangular in plan. The control tower is a 10-story building and approximately square in plan. The base operations building is constructed of concrete. The control tower is constructed of concentrically braced frames with precast concrete cladding. All interior walls are nonstructural. These two buildings are separated by a 4 1/2-in. joint. In addition to finding the facility in good condition during the field inspection, researchers verified the existence of the seismic joint.

The vertical load carrying system of the base operations building consists of a roof slab carrying the load to reinforced concrete bearing walls and reinforced concrete interior columns. The walls and columns deliver the load to continuous strip footings. The lateral load resisting system consists of a reinforced concrete diaphragm that transfers the load to the reinforced concrete shear walls in both the transverse and longitudinal directions.

In the lateral analysis of the base operations building, the interior columns were not considered to resist seismic loading. Researchers assumed that due to the greater stiffness of the walls relative to the columns, the walls would attract the load until their failure and then the interior columns may act with the roof beams as frames in resisting lateral load.

The vertical load carrying system of the control tower consists of the roof slab and intermediate steel deck with lightweight concrete fill at the floors. These deliver the load to beams and columns of the steel braced frames. Lateral loads are resisted by the concentrically braced steel frames at each story and the belled piers at the foundation. Between the first and second floors of the tower, the centroids of the V and chevron braces are not concentric. This eccentricity, if large enough, can cause premature yielding of the beams and could thereby reduce their ultimate capacity. Also the V and chevron braces will experience problems if one of the braces buckle. Large vertical deflections could occur in the floor beam at its joint with the braces; this could lead to local failure of the gravity load system.

Building 1086 - Command Post

The command post is a large one-story building with a partial basement and an irregular plan. The average building height is approximately that of a two-story structure. In addition, there are two penthouse mechanical rooms; one with a floor slab at approximately roof level and another with a floor below roof level. A parachute drying tower also rises approximately six stories above the floor slab. The

primary construction of the original building is precast concrete columns, precast prestressed double T beams and precast walls. A 200-by 80-ft section addition is seismically independent and will be referred to throughout this analysis as area A-VI. In the seismically independent area, the primary construction is X-braced steel columns and beams with exterior precast concrete panels.

The vertical load carrying system of the main building consists of precast prestressed concrete double T beams that carry the roof loads to precast beams or, in some cases, directly to load bearing precast walls. Beam loads are transferred to precast concrete columns; columns are supported by spread footings. Vertical wall loads are then transferred to the continuous strip footings.

The lateral load resisting system of the main building consists of precast and cast-in-place concrete shear walls in the transverse and longitudinal directions as well as columns that lie within the shear plane of the wall and are positively connected to the shear walls. All other columns were assumed to not resist lateral loads. Structural interior walls, as well as exterior walls, are positively connected to the roof slab, the roof beams, and to any columns in the wall's plane and were assumed to act in resisting lateral forces. Precast concrete walls are connected to the roof diaphragm and foundation by reinforcing steel and welded plates; the capacity of these connections was evaluated. Researchers assumed that all roof panels, beams, and columns were sufficiently connected to form a diaphragm to transfer all loads. Based on the survey, the building appeared to be in good condition.

In area A-VI, vertical loads are transferred from the metal deck through purlins and wide-flange beams to supporting wide-flange columns along the perimeter of the area and down the transverse centerline of the building. The columns are supported by reinforced concrete spread footings.

The lateral load system in area A-VI consists of a metal roof deck transferring the load to a combination of precast concrete shear walls and steel braced frames. Similar to the original structure, the shear capacity of the precast concrete walls was based on the capacity of the welded connection between the walls and the foundation. This structure was also in good condition.

Building 1200 - Air Refueling Squadron Operations

The air refueling squadron operations building has one story above ground and a full basement, and is rectangular in plan. In addition, it has sloping entrances and a mechanical room attached to the structure at the basement level.

The vertical load carrying system consists of open-web steel joists that carry the roof loads to interior and exterior bearing walls. Reinforced concrete and reinforced concrete masonry unit (CMU) walls in the basement carry the load to continuous strip footings. The lateral load resisting system consists of concrete roof and floor diaphragms and CMU shear walls in the transverse and longitudinal directions.

Details for the mechanical room were not available, therefore, it was not considered in any part of the analysis. In the lateral analysis, the masonry entrances to the structure at the first floor were not considered to provide any shear resistance. The perimeter reinforced CMU walls and center reinforced CMU walls at the first floor alone were considered to provide shear resistance. Many of the CMU interior partitions may provide additional lateral resistance but their connection to the first floor slab is not clear from the available drawings and, therefore, they were not assumed able to transfer shear.

Building 2145 - Continuous Processing

The continuous processing building is comprised of two structures, the operations building and the power building. They are separated by a 2-in. expansion joint. The first floor of both buildings is located 3 ft 11 in. above grade; each has a basement. Although the power building has only one floor, it is approximately two stories tall. It is essentially rectangular in plan. The operations building has four stories and is rectangular in plan.

In the operations building, vertical loads are carried by a concrete flat slab system to concrete columns capped by drop slabs. Columns are continuous throughout the height of the building. They are supported at their base by a concrete mat foundation. Lateral load resistance is provided by the concrete floor diaphragms, the concrete shear walls at the perimeter of the building, and the frame action of the concrete flat slab and columns. Overturning is resisted by the concrete mat foundation. Because the two buildings are seismically separated by a 2-in. expansion joint, they were assumed to resist lateral loads independently.

The vertical load carrying system of the power building is composed of a concrete roof slab with integral beams, and reinforced concrete frames. Lateral loads are resisted by the concrete roof diaphragm and the concrete frames, which extend to the basement, and the perimeter concrete shear walls. Overturning is resisted by a reinforced concrete mat foundation.

Building 2340 - Officers Club

The Officers Club is sited on a hill and the structure is stepped into the slope of the hill. The primary construction of the one-story west half of the building is perimeter CMU walls; in the two-story area, the primary construction is reinforced concrete frames. The second floor of the structure is also two stories high in this region. The roof loads are supported by double T concrete beams. These loads are transferred to CMU bearing walls and the reinforced concrete columns and then to the foundation. The gravity load of the intermediate floor and its superimposed loads are carried by CMU walls.

The lateral load resisting system consists of CMU shear walls in the longitudinal and transverse directions. In the lateral analysis of the building, researchers did not consider the concrete frames as resisting seismic loads. Detailing of the beam-column connections of the frames does not appear to have considered moment resistance due to lateral loads. Additionally, the CMU infill walls are not seismically separated from the frames and, due to their greater stiffness, will attract the load until their failure.

Building 2418 - Gymnasium

The main portion of the gymnasium is approximately three stories high and rectangular in plan. This area includes the later addition of a racquetball court at the building's northeast corner. Adjacent to the main portion of the building is a handball and office area which is 1-1/2 stories high. The primary construction of the building is exterior and interior CMU walls. The vertical load carrying system consists of open-web joists and wide-flange steel beams that carry the roof loads to rigid steel frames, steel pipe columns, and interior and exterior CMU load bearing walls. The foundation is constructed of continuous reinforced concrete strip footings. The lateral load resisting system consists of CMU shear walls in the longitudinal and transverse directions. In the lateral analysis of the building, the steel frames are not considered to resist seismic loads. The interior and exterior CMU bearing walls are stiffer and were assumed to resist the total earthquake load.

Building 2434 - Post Exchange

The post exchange is approximately 1-1/2 stories tall and basically rectangular in plan. It is constructed primarily of structural steel, CMU, and tilt-up concrete. The vertical load carrying system consists of wide-flange steel beams that carry roof loads to interior steel columns and exterior bearing CMU, and tilt-up concrete walls. The lateral load resisting system consists of a metal deck diaphragm that transfers loads to CMU shear walls, and tilt-up concrete shear walls with composite concrete/steel pipe exterior pilasters in the longitudinal and transverse directions.

In the lateral analysis, researchers assumed that the connection between the tilt-up concrete walls and foundation was insufficient for transferring shear. The apparent connection between the pilasters and the tilt-up walls is via shear friction. Researchers therefore assumed that the shear capacity of the building was governed by the capacity of the pilasters alone and the CMU walls.

Building 2459 - Commissary

The commissary is constructed of two structures. There are no drawings from which to analyze the original base supply and equipment warehouse. The second structure, commissary store, is a one-story building approximately two stories high and is basically rectangular in plan. Gravity loads are supported by the wide-flange steel beams and open-web steel joists. These carry the loads to steel pipe columns or load bearing exterior tilt-up concrete walls and cast-in-place concrete pilasters.

The lateral load is resisted by the metal deck diaphragm that transfers loads to the tilt-up concrete walls in both the longitudinal and transverse directions. These walls are tied to the floor slab by threaded inserts and to the columns by welded angles. The load is ultimately resisted by the cast-in-place concrete pilasters. While it is not clear that the original base supply and equipment warehouse and added commissary store, dairy cooler, and freezer are separated from one another, researchers performed a simplified analysis of the commissary store to gain a better understanding of the seismic resistant capacity of the additions. A more detailed analysis is required as no plans were available to analyze the original structure. If the buildings are structurally connected, the influence of the buildings on one another cannot be assessed from this analysis.

Building 2474 - Theater

The theater is comprised of a two-story tall section housing the main theater, a two-story section for the concessions and projection room, and a one-story storage area. Its configuration is rectangular and the construction is primarily of CMU. The vertical load carrying system consists of steel joists and beams, or steel long span trusses that span to steel wide-flange columns in the masonry walls. The lateral loads are transferred from a steel deck roof diaphragm to CMU shear walls at the exterior of the building and one interior wall in the transverse and longitudinal directions. Researcher based the capacity of the building on the shear capacity of these walls. A field inspection showed the building to be in good condition.

Building 2490 - Dining Hall

The dining hall is a one-story rectangular building. The primary construction materials are CMU walls, concrete columns, steel beams, steel joists, and a space truss. The vertical load carrying system consists of open-web steel joists and wide-flange steel beams or space trusses which carry the roof loads to concrete columns, steel pipe columns, and interior and exterior load bearing walls. Continuous

reinforced concrete strip footings form the foundation beneath the walls. Pad footings support the weight of concrete and steel pipe columns.

The lateral loads are transferred from the steel deck roof diaphragm to reinforced CMU walls in the transverse and longitudinal directions. In this simplified analysis, researchers assumed that the concrete columns did not resist lateral load. The interior and exterior CMU walls were assumed to resist the total lateral load.

Building 5700 - Hospital

The hospital was constructed in two stages. The main structure and nursing wing were built first. The clinic and nursery addition were added later. Each building is one story. The main structure is rectangular in plan with a small extension that connects it to the nursing wing. The nursing wing is also rectangular in plan; the clinic is F-shaped in plan. For analysis, the nursery addition was considered part of the main structure. The primary construction material for all portions of the facility is reinforced concrete. The vertical load carrying systems of the main structure and nursing wing are constructed of a reinforced concrete two-way slab supported by reinforced concrete columns, with square column capitals at the interior of the structure and reinforced concrete load bearing walls at the perimeter. Spread footings carry the load of the columns, and strip footings support the weight of the walls.

The lateral force resisting systems of the main structure and nursing wing are composed of a reinforced concrete diaphragm and reinforced concrete shear walls at the perimeter of the building portions. The columns were assumed to not resist lateral load. The buildings were analyzed independently as they are seismically separated from one another by a 2-in. open joint. The main structure is also seismically separated from the clinic by a similar joint.

Vertical loads of the clinic are supported by wide-flange steel beams that carry the roof load to steel pipe columns at the interior of the building, and perimeter concrete walls. These deliver the load to reinforced concrete footings. Lateral loads are resisted by the reinforced concrete walls of the clinic.

Building 5800 - Noncommissioned Officers' (NCO) Club

Several additions were made to the original construction of the NCO club. These additions appear to be integral with the original structure. The entire structure is constructed of CMU walls and is one story in height. The building is extremely irregular in plan and has many reentrant corners. The vertical load carrying system consists of open-web steel joists and wide-flange steel beams that carry roof loads to interior and exterior bearing walls or steel pipe columns. Loads at the bearing walls are carried by continuous strip footings.

The lateral load resisting system consists of CMU shear walls in both the longitudinal and transverse directions. It appears from the drawings that the building additions were positively tied to the original structure such that they would work together to resist lateral load. Shear walls are well distributed throughout the building. Field inspection of the building confirmed the construction of the structure and that the building was in good condition.

A summary of the buildings analyzed at Beale AFB is provided in Table 15.

Table 15

Description of Buildings Analyzed at Beale AFB

Bldg No.	Year Built	No. of Stories	Total Area	Type	Lateral Force Resisting System
1025	1964	1	78,000	Steel/ CMU/ Precast	Precast C shear walls
1029	1961	2	16,444	Steel/RC /Precast	Precast/RC shear walls
1060(BO)	1958	1	9,343	RC	RC shear walls
1060(CT)	1958	10	4,030	Steel	Steel braced frames
1086(main).	1958	1	172,408	RC/ Precast	RC/Precast C shear walls
1086(A-VI)	1959	1	14,330	Steel/ Precast	Precast C shear walls/Steel braced frames .
1200	1959	1	8,424	Steel/CMU	CMU shear walls
2145(main)	1959	4	41,556*	RC	RC shear walls
2145(pwr)	1959	1	10,500	RC	RC frames
2340	1959	2	7,838	RC/CMU	CMU shear walls
2418	1966	1	23,300	Steel/CMU	CMU shear walls
2434	1971	1	42,100	Steel/RC	CMU/RC shear walls
2459	1972	1	22,150	Steel/RC	RC shear walls
2474	1973	1	10,490	Steel/CMU	CMU shear walls
2490	1981	1	15,970	Steel/RC/ CMU	CMU shear walls
5700	1959	1	65,319	RC	RC shear walls
5800	1959	1	21,916	CMU	CMU shear walls

* Only the first floor of this structure requires upgrading.

Castle AFB Building Descriptions

Building 175 - Weapons System Training Center

The weapons system training center was constructed in a symmetrical I-configuration. The wings are two stories with a partial third story; the center is two stories tall. The vertical load carrying system at the roof of the wings is double T beams supported by precast concrete inverted T beams that span between concrete columns or between load bearing perimeter walls. At the intermediate floors, vertical loads are carried by a reinforced concrete joist slab system with integral beams. Columns support the beams and carry the loads to spread footings. Precast concrete walls are supported by continuous grade beams. Lateral loads are resisted by a combination of the precast concrete perimeter walls with cast-in-place pilasters acting in shear, and the reinforced concrete frame formed by the beam-slab and columns in both the longitudinal and transverse directions of the wings and center portions of the building.

During a field inspection of the building, researchers observed that the facility housed important flight simulation equipment and associated computer facilities. This equipment has a high potential for loss in a severe earthquake. The building appeared to be in good general condition.

Seismic joints separate the middle section of the building from the two wings. The three portions of the building were assumed to act independently for the lateral analysis. Dry pack grout forms the connections between the panels and the foundation; no reinforcement is present in the shear plane. Panels are connected to cast-in-place pilasters by #4 dowels at 12 in. on center (o.c.). Although the precast walls may resist shear, they were not considered in this analysis. All lateral load was assumed to be resisted by the reinforced concrete moment frames. Seismic capacity calculations were based on the shear strength of the concrete columns.

Building 360 - Dining Hall

The dining hall is basically rectangular in plan and one story in height. The building consists of perimeter concrete frames infilled with CMUs, and interior concrete and CMU walls. The vertical load carrying system consists of wide-flange steel beams, purlins, and a metal deck diaphragm which carry the roof loads to interior and exterior bearing walls. The foundation is constructed of continuous reinforced concrete strip footings under the walls and spread footings beneath independent columns. An inspection of the building found it to be in good condition.

The metal deck diaphragm transfers lateral loads to interior and exterior shear walls in both the longitudinal and transverse directions. In the lateral analysis of the building, the perimeter concrete frames were not considered to resist seismic loads. Detailing of the beam-column connections of the frames does not appear to have considered moment resistance due to lateral loads. Additionally, the CMU infill walls are not seismically separated from the frames and, due to their greater stiffness, will attract the load until their failure. These walls and the reinforced concrete walls were assumed to resist the total lateral load.

Building 752 - Commissary

The commissary is a one-story reinforced concrete and concrete masonry block building. It is irregular in plan. The vertical load carrying system consists of wide-flange steel beams and purlins carrying roof loads to interior and exterior CMU and concrete bearing walls. Continuous concrete strip footings form the foundation. A metal deck diaphragm transfers lateral loads to reinforced concrete frames with CMU infill and reinforced concrete shear walls in both the transverse and longitudinal directions.

Field inspection of the building confirmed the construction of the structure and the building was found to be in good condition. No drawings were available for the warehouse addition to the commissary, therefore it could not be analyzed.

In the lateral analysis of the building, researchers assumed that the loads were resisted initially by the CMU walls and reinforced concrete walls due to their greater stiffness as compared to the reinforced concrete frames.

Building 759 - Base Exchange

The base exchange is rectangular in plan and primarily of steel construction. Perimeter walls are of CMUs with a parapet extending approximately 2 ft above the roof framing. The vertical load carrying system consists of purlins spanning to tapered steel girders. The girders are simply supported by CMU pilasters and steel pipe columns. Reinforced concrete canopies cover the main and side entrances to the building. The canopies are supported by reinforced concrete columns. The lateral load resisting system is composed of a steel deck roof diaphragm that transfers the load to the CMU perimeter shear walls. The pilasters and steel framing were not assumed to resist any lateral load. The building was found to be in good condition during a field inspection.

Building 786 - Theater

The theater is comprised of a two-story section housing the main theater, a two-story section for the concessions and projection room, and a one-story storage area. The building is essentially rectangular in plan. The vertical load carrying system is composed of steel trusses transferring roof loads to the precast concrete bearing walls. Loads at the bearing walls are carried by continuous concrete strip footings. Lateral loads are resisted by a gypsum concrete diaphragm that transfers load to concrete pilasters and precast concrete wall panels at the perimeter of the building in both the transverse and longitudinal directions. Only a bed of dry-pack grout within a shear key connects the precast panels to the foundation; no reinforcing is present. The wall panels are additionally connected to the concrete pilasters by #5 bars at 15 in. o.c. and a shear key.

For the lateral analysis, the shear capacity of the precast concrete wall panels was assumed to be limited by the connection between the walls and the foundation. The capacity of the walls was therefore taken to be the shear friction capacity of the dry-pack grout in contact with the concrete. Based on the field survey, the building appeared to be in good condition.

Building 1212 - Law Enforcement Center

The law enforcement center originally functioned as a barracks. The building is rectangular in plan and three stories tall. Primary construction is reinforced concrete. The vertical load carrying system consists of reinforced concrete floor and roof diaphragms that distribute loads to reinforced concrete beams. These span to reinforced concrete columns supported by spread footings. The lateral forces are transferred by the concrete floor and roof diaphragms to concrete frames. Frames are aligned along each longitudinal perimeter face and two interior lines for lateral resistance in the longitudinal direction. Two reinforced CMU end walls resist lateral loads in the transverse direction. A field inspection of the building found it to be in good condition.

In the lateral analysis of the building, researchers evaluated the shear capacity of the reinforced concrete frames in the longitudinal direction and the masonry walls in the transverse direction.

Building 1230 - 93rd Bombardment Wing Command Post

The 93rd Bombardment Wing command post consists of several seismically separated buildings. The original portions are the wing headquarters (WHQ) and Target Intelligence Training Building (TITB). The WHQ is composed of two one-story rectangular units. The TITB is separated from the WHQ by a 1-in. expansion joint. The TITB vault and MDPS additions are each less than 3000 sq ft and are separated from the balance of the structure by 6-in. and 2-in. expansion joists, respectively. Due to their small area, they were not analyzed.

The vertical load carrying system of the WHQ consists of wood sheathing and 2- by 8-in. joists supported by wide-flange steel beams. The beams are supported by CMU bearing walls. Reinforced concrete strip footings support interior and exterior walls. The lateral load resisting system is composed of CMU shear walls.

Building 1260 - Jet Engine Maintenance

The jet engine maintenance facility was originally constructed as a reinforced concrete frame structure with windows in most frame openings. Approximately 75 percent of the windows were replaced with CMU infill. The building is rectangular in plan and has one story, approximately 31 ft tall. The vertical load carrying system of the building is metal roof deck and steel purlins simply supported by steel trusses. Trusses are simply supported by wide-flange columns along interior and perimeter lines. Spread footings distribute loads from the columns to the soil. Lateral loads are transferred by the metal deck diaphragm to the reinforced concrete frames at the perimeter of the structure. There are, therefore, two frames in each direction. During the field inspection researchers noted that very heavy engines are lifted by cranes within the structure. The building appeared to be in good condition.

The rapid lateral analysis evaluated the shear capacity of the reinforced concrete frames. The CMU walls restrain the columns from deflecting freely and column heights were modified to account for this stiffening. The CMU walls are not connected to the roof diaphragm and, therefore, were not considered to carry lateral loads.

Building 1325 - Field Maintenance Squadron

The field maintenance squadron building is rectangular, one-story, and built of reinforced CMU. The vertical load carrying system consists of steel beams carrying loads from the roof to pilasters in the bearing walls or steel pipe columns along the centerline of the building. Lateral load resistance is provided by a plywood diaphragm that transfers loads to CMU shear walls in both the longitudinal and transverse directions. During the field inspection, researchers noted that along the north elevation of the building, the third opening had been infilled with CMU and a door. The replacement of the original flat roof with a sloped roof was also noted. The building appeared to be in good condition.

Lateral analysis of the building evaluated the shear capacity of the reinforced CMU walls. All shear walls are punctuated by openings, significantly reducing their shear area. This is particularly critical in the transverse direction on the south face of the building.

Building 1335 - Armament Maintenance Squadron

The armament maintenance squadron building is rectangular in plan and one story tall. A small addition is attached to it and a penthouse is located on top of the original structure. The vertical load carrying system consists of a concrete-topped steel deck supported by open-web steel joists and steel beams. These are simply supported by CMU bearing walls. Reinforced concrete strip footings are located under interior and exterior walls. Interior columns are supported by reinforced concrete spread footings. The lateral load resisting system is concrete moment frames with CMU infill and CMU shear walls. During a survey of the building, researchers identified cracking in the north wall at one-third points from the end of the building. They also observed cracking in the north garage area on the east face of the building. The bearing walls appear to be separating from the pilasters in the area of the new addition.

For the lateral analysis, researchers assumed that the weight of the penthouse was lumped at the roof and that the concrete moment frames did not resist lateral load until the CMU walls had failed. The CMU walls are much stiffer than the frames and will attract the load before the frame.

Building 1340 - Base Operations

This facility is comprised of a flight tower and a main operations building. The tower is constructed of reinforced concrete. It is nine stories tall, with an overall height of 93 ft, and is rectangular in plan. The main building is primarily of wood construction, and varies from one to two stories. The vertical loads of the main building are carried by wood joists to wood bearing walls. The walls are supported by reinforced concrete strip footings. In the tower, reinforced concrete slabs carry all vertical loads to reinforced concrete bearing walls and the mat foundation. The lateral load resisting system in the main building is provided by the wood diaphragms and wood shear walls. In the tower, lateral loads are resisted by shear action of the concrete diaphragms and reinforced concrete walls. During the field survey, researchers found the building to be in good condition.

The lateral resistance of the main building was not evaluated because wood buildings have historically performed well in earthquakes. This building is seismically separated from the tower, therefore the tower was analyzed independently. The shear capacity of the tower's reinforced concrete walls was evaluated.

Building 1344 - Fire Station

The fire station is one story and is rectangular in plan. Two additions were made to the building; these are structurally connected to the original building. The vertical load carrying system consists of wood joists supported by steel beams or masonry bearing walls or perimeter concrete frames. The steel joists are supported by masonry bearing walls. Continuous concrete strip footings transfer vertical loads to the soil. The lateral load resisting system is provided by a wood diaphragm that transfers the load to masonry and concrete shear walls and concrete perimeter frames; one in the transverse direction and one in the longitudinal direction.

The lateral analysis evaluated the capacity of the combined shear wall and frame resisting system in both the longitudinal and transverse directions.

Building 1350 - Maintenance Hangar

The maintenance hangar is a one-story rectangular building. The roof diaphragm is a 22-ft tall steel truss supported by four shop structures. The three-truss arches spanning between shops are pin-connected at center span. Each shop tower is 42 ft tall and constructed of a three-story steel braced frame with concrete floor and roof. Vertical loads are carried by a steel truss and arch system supported by the columns of the three shop towers. The foundation of the columns consists of reinforced concrete spread footings. The lateral load resisting system consists of the steel braced frames of the towers. The field survey showed the building to be in good condition.

A rapid seismic evaluation consisted of calculating the shear capacity of the braced frames in the longitudinal and transverse directions.

Building 1360 - Base Supply

An original warehouse and an extension, approximately the area of the original structure, comprise the base supply. The building is rectangular in plan, and two stories tall with a main floor and a mezzanine level along the southwest side of the building. The perimeter walls are cast-in-place concrete in the original warehouse and precast concrete wall panels and concrete pilasters in the warehouse extension area. The vertical load carrying system for the original warehouse consists of a wood deck over wood purlins and laminated wood girders that carry the loads to exterior bearing walls and columns. These are supported by spread and strip footings. In the warehouse extension, wood purlins and wide-flange steel beams and girders carry vertical loads to exterior bearing walls and columns, which then carry the loads to the spread and strip footings.

A field investigation indicated that there are many very tall storage shelves in both portions of the building. These appear to be adequately braced to resist seismic loads though no analysis was done on these nonstructural components. While the shelves may remain standing after an earthquake, the contents of the shelves will be cast onto the floors as there are no provisions to confine the contents under lateral load.

The lateral analysis of the building assumed that the original warehouse and extension act as one building. Details show that the extension is positively connected to the original structure. The capacity of the structure was based on the shear capacity of the reinforced concrete walls in the original warehouse and the capacity of the dry grout connection of the precast shear walls in the extension. The reinforced concrete pilasters of the extension were also assumed to resist lateral load.

Building 1532 - Precision Measurement Equipment Systems

The precision measurement equipment systems facility was constructed of reinforced CMU. It is a one-story building approximately rectangular in plan. The vertical loads are supported by open-web steel truss joists. These transfer the loads to the bearing walls and continuous strip footings. Lateral loads are resisted by the steel deck diaphragm at the roof, which transfers load to interior and perimeter CMU shear walls. A field inspection of the building found it to be in good condition.

In the lateral analysis of the building, researchers evaluated the capacity of the CMU walls.

Building 1540 - Squadron Operations

The squadron operations building was primarily constructed of CMU. The structure has two stories; the second floor covers the north two-thirds of the first floor. The vertical load carrying system is a wood deck, and wood joists spanning to bearing walls and steel beams. Steel beams are supported by steel pipe columns and CMU wall pilasters. All walls are supported by continuous footings; pipe columns are supported by a thickened slab. The lateral force resisting system is provided by wood diaphragms and CMU perimeter shear walls. A field inspection of the building showed the structure to be in good condition. In the lateral analysis of the building, researchers evaluated the shear capacity of the reinforced CMU walls.

Building 1582 - SAC Alert Facility

The SAC alert facility has two stories; the first story is surrounded by an earth berm. Corridors, sloped to grade, extend from the basement to the outside. Basement walls are cast-in-place concrete. Above-grade construction is CMU. The vertical load carrying system consists of concrete slabs supported by long span steel open-web joists at the roof and floor. Joists are supported by CMU or concrete bearing walls. Lateral loads are resisted by concrete floor and roof diaphragms that transfer loads to reinforced CMU shear walls.

Lateral analysis of the building evaluated the shear resistance of the perimeter reinforced CMU walls and the center reinforced CMU wall on the interior of the building at the above-grade floor. There are many CMU interior partitions but their connection to the first floor slab is not known, therefore they were not assumed to be capable of transferring lateral load.

Table 16 describes the buildings analyzed at Castle AFB.

March AFB Building Descriptions

Building 300 - Hangar/Warehouse

The original hangar is rectangular in plan. Office annexes were added the full length of the original building shortly after its construction. At its maximum height, the building is approximately five stories tall. The height of the roof diaphragm in the office area varies from 1 to 1-1/2 stories. Primary construction material of the hangar is structural steel.

The vertical load carrying system of the original hangar consists of steel purlins spanning between transverse steel roof trusses. These are supported by wide-flange steel beams at the perimeter of the original hangar. Columns rest on concrete pedestal footings. Lateral loads are transferred by the diagonal rod bracing in the plane of the upper chord of the roof truss to diagonal bracing in the plane of the original hangar walls. The reinforced concrete walls of the office annexes also resist lateral load. The building appeared to be in good condition when inspected.

In the lateral analysis of the structure, researchers assumed that the total lateral load was resisted by the reinforced concrete perimeter walls of the office annexes in both the longitudinal and transverse directions. Some rod bracing in the plane of the walls of the original structure had been removed. Additionally, it is not clear how the new reinforced concrete walls at the perimeter of the original hangar are connected to the roof diagonal bracing at their tops and the original terra-cotta walls at their bases. It is unknown if lateral load may be transferred to the walls and then to the foundation.

Table 16

Description of Buildings Analyzed at Castle AFB

Bldg No.	Year Built	No. of Stories	Total Area	Type	Lateral Force Resisting System
175	1978	2	93980	Precast C/RC	RC frame
360	1958	1	14430	CMU/RC	CMU/RC shear walls
752	1959	1	54120	RC/CMU	CMU/RC shear walls
759	1978	1	52480	CMU	CMU shear wall
786	1956	2	9070	Precast/ RC	Precast C shear walls
1212	1953	3	25230	RC	RC frame/CMU shear walls
1230	1955	1	29130	CMU	CMU shear walls
1260	1955	1	32990	RC/CMU	RC frame
1325	1955	1	5380	CMU/Steel	CMU shear walls
1335	1955	1	30860	RC/CMU	CMU shear walls
1340	1952	9	20910	RC	RC shear walls
1344	1954	1	16140	CMU	CMU shear walls /RC frame
1350	1954	1	191580	CMU/Steel Steel	Steel braced frame
1360	1952	1	72990	RC	RC shear walls
1532	1961	1	12090	CMU	CMU shear walls
1540	1956	2	11555	CMU	CMU shear walls
1582	1960	2	19050	CMU	CMU shear walls

Building 465 - Physical Fitness Center

The physical fitness center is a one-story building rectangular in plan. The maximum building height is approximately four stories at the roof peak, sloping to two stories in height at the eaves. All interior and exterior walls are clay tile masonry units. The roof is supported by large, deep trusses on steel columns.

The vertical load carrying system consists of steel roof trusses that carry vertical loads to exterior steel columns. The columns carry the load to spread footings. Top and bottom chords of the steel roof trusses are braced to form a roof diaphragm that resists lateral loads. Forces are then transferred to clay tile shear walls in the transverse and longitudinal directions, and then to continuous strip footings. The facility appeared to be in good condition when inspected.

Many simplifying assumptions were made to analyze this structure. The shear capacity of clay tile is not explicitly known, and the availability of design details was limited. Capacity of the clay masonry was assumed to be the same as that of concrete masonry, and the shear area was assumed to be the bedded

net area of a typical 12-in. clay unit. The masonry was assumed to be unreinforced and ungrouted. Diagonal steel braces were observed during the site inspection and are indicated on the drawings, but the number and location of the rods identified in the site inspection does not match that shown on the drawings. It is possible that the steel framing may provide some lateral load resistance in conjunction with the masonry, but it was not considered as resisting load in this preliminary analysis.

Building 651 - Recreation Center

The recreation center is a two-story building, irregular in plan. Steel joists, beams, and girders carry the roof loads to the exterior and interior CMU bearing walls and columns. Lateral loads are resisted at the roof by a reinforced gypsum diaphragm; the floor diaphragm is a reinforced concrete slab, joist, and beam system. These transfer loads to CMU walls in the transverse and longitudinal directions. Exterior and interior walls are supported by reinforced concrete strip footings; spread footings support the columns. Results of the building survey showed the building to be in good condition.

In the lateral analysis of the building, it was assumed to be one-story as the area of the second floor roof is very small relative to the total building area.

Building 760 - Theater

The theater is comprised of a two-story section housing the main theater, a two-story section with a concession area on the first floor and the projection room on the second, and a one-story storage area. The building is basically rectangular in plan with primary construction materials of CMU and glazed structural units (GSU).

The vertical load carrying system consists of steel joists carrying the roof loads to interior and exterior bearing walls. The loads are then delivered to the continuous strip footings. The lateral forces are resisted by the steel deck diaphragm with lightweight concrete fill, which transfers loads to the shear walls composed of GSU. The building appeared to be in good condition.

In the lateral analysis of the building, researchers assumed the structural capacity of the GSU to be comparable to that of CMU. The building was evaluated as a one-story building with loads being transferred from the high roof area to the low roof area through shear in the walls.

Building 960 - Commissary

The commissary is a one-story building. It is composed of two newer structures built around an existing warehouse, originally Building 956, now included as a part of Building 960. Maximum building height is approximately two stories. The warehouse is a one-story building and has a maximum height approximately equal to three stories. Primary construction of the new buildings is CMU walls with internal steel framing; the original warehouse is constructed of exterior insulated metal wall panels and internal steel framing.

The vertical load carrying system of the commissary store consists of steel beams and open-web joists that carry the roof loads to steel columns, interior and exterior CMU walls, and CMU pilasters. Continuous, reinforced concrete strip footings support the loads transferred by the bearing walls. Pad footings support the load acting on the steel columns. The vertical load carrying system of the commissary warehouse consists of metal roof panels and steel purlins that transfer gravity loads to rigid steel frames. Concrete pad footings support the loads of the frames. The lateral load resisting system of

the commissary store consists of CMU shear walls in the longitudinal and transverse directions. The lateral load resisting system of the commissary warehouse is provided by braced frames in both the longitudinal and transverse directions.

The buildings are separated by 2-in. seismic joints. They were therefore analyzed separately. The rigid frames of the commissary warehouse were not considered capable of resisting lateral load as the columns do not form a moment connection with the foundation.

Building 962 - Dining Facility

The dining hall is a single-story building. Architectural renovation was carried out several times after initial construction but there are no records of the structural modifications associated with this work. The building is irregular in plan with many reentrant corners. The vertical load carrying system consists of wood rafters and wide-flange steel beams that carry the roof loads to steel columns and reinforced concrete columns. Pad footings support the columns. Continuous strip footings support the weight of the CMU walls. The lateral load resisting system consists of a plywood roof diaphragm and CMU shear walls in the longitudinal and transverse directions. No problems were noted during the building inspection.

In the lateral analysis of the building, the steel and concrete columns were not considered part of the system. Interior and exterior CMU walls were assumed to resist the total lateral load. The large window expanses on the east and west elevations of the main dining hall have significant influence on the seismic capacity of the structure.

Building 1220 - Base Operations and Control Tower

The base operations and control tower are two separate structures with essentially rectangular plan configurations. The base operations building is two stories with the second floor smaller and centered over the first. Primary construction of this building is reinforced concrete and CMU. The control tower is situated immediately next to the front of the building. It has 12 stories above the ground floor. It is constructed of structural steel and precast concrete.

The vertical load carrying system of the base operations building consists of a lightweight concrete slab on metal decking and open-web steel joists at the roof, which span to CMU walls, and a reinforced concrete floor slab. This is supported by concrete beams spanning to concrete columns or CMU walls. Continuous, reinforced concrete strip footings run beneath all walls. The lateral load resistance of the building is provided by the concrete and metal roof deck diaphragm that transfers loads to CMU shear walls. The concrete floor slab redistributes these loads to the CMU walls below.

The vertical load resisting system of the control tower is provided by structural steel framing at the roof and intermediate floors with reinforced concrete slabs at selected floors and structural steel columns and intermediate girders. These carry the loads to bearing walls at the first floor, which are supported by a concrete mat foundation. Lateral loads are resisted by precast concrete panels that carry loads to the reinforced concrete shear walls through welded steel connections. During the building inspection, researchers noted the lack of separation between the tower and the south end of the base operations building.

Building 1223 - Fire Station

The fire station's basic structure is similar in configuration to the Fire Station at Castle AFB with a few exceptions. Three additions were made to the building: a dining area, a garage, and a technical service area. Later, two other spaces were constructed onto the original building. The entire structure has only one story, however, the height varies between approximately one and two stories. The tower is about four stories in height. The building is irregular in plan.

The vertical load carrying system consists of open-web steel joists that carry roof loads to the exterior and interior bearing walls. Reinforced concrete spread footings are located under interior columns and strip footings are under all walls. The lateral load resisting systems consists of a steel deck diaphragm and gypsum formboard at the roof, and CMU and concrete walls in the transverse and longitudinal directions. No structural problems were identified from the field survey of the building.

In the lateral analysis of the building, researchers noted that the high bay garage is seismically separated from the building. It is 1261 sq ft in area and, therefore, was below the established limit of 4000 sq ft and was not analyzed. While there are many different roof levels to the structure, lumped masses were assumed at elevations of 22 ft and 12 ft. The transfer of shear from the tower above 22 ft to the main structure was also evaluated.

Building 1305 - Readiness Crew

The readiness crew building was constructed in 1958. It is a one-story building basically rectangular in plan with a full basement. The overall building dimensions are 114 ft 0 in. by 78 ft 0 in. Additionally, entrances and a storage shed are attached to the structure. The maximum building height is approximately 14 ft. The vertical load carrying system consists of open-web joists that carry the roof loads to interior and exterior bearing walls. Reinforced concrete and reinforced masonry walls in the basement carry the load to continuous spread footings. The lateral load resisting system consists of a reinforced gypsum concrete roof diaphragm over gypsum formboard and CMU shear walls in the transverse and longitudinal directions. The building appeared to be in good condition when inspected.

Detailed floor plans of the first floor of the structure were not available, however, the main structural system is identical to structures previously analyzed at Castle and Beale AFBs. It appears that the floor plan, with respect to partition walls, is slightly different and that the March AFB structure is 6 ft longer in the longitudinal direction. For the sake of this analysis, and because there is a lack of complete structural plans, researchers assumed the capacity of this structure is the same as the similar readiness crew buildings analyzed at Castle and Beale AFBs.

In addition, details of the mechanical room were not available, therefore it (including a masonry chimney) was not considered in any part of the analysis. The masonry entries to the structure also were not considered to provide any shear resistance.

Building 2300 - Base Contracting

Base contracting is primarily a one-story building with an air conditioning penthouse and an antenna penthouse. The building plan is symmetric about the transverse axis except for the boiler room attached to the north wall. The balance of the structure is rectangular in plan with an inset area for the loading dock. Most structural walls are concrete frame with masonry infill.

The vertical load carrying system consists of a reinforced concrete slab and metal deck supported by steel joists that carry vertical loads to interior and exterior bearing walls and beams supported by columns. The bearing walls carry the load to continuous strip footings, while the columns carry the load to spread footings. The lateral load resisting system consists of a horizontal diaphragm of reinforced concrete and a metal deck that transfer load to CMU shear walls and concrete frames with reinforced masonry infill in the transverse direction, and concrete frames with reinforced masonry infill in the longitudinal direction. The building appeared to be in good condition during the inspection.

The building was evaluated as a multistory structure due to the distribution of loads at the penthouses. Lateral loads were assumed to be resisted by the CMU infill walls as these are stiffer than the reinforced concrete frames and would resist the load until their failure.

Building 2303 - Maintenance Hangar

The maintenance hangar is the same as Building 1350 at Castle AFB. Therefore, the results of the Castle AFB analysis were used to determine the seismic vulnerability of Building 2303. It is a one-story rectangular building. The roof structure is a 22 ft. 0 in.-high steel truss supported by four shops. The three truss arches spanning between shops are pin-connected at center span. Each shop is 42 ft tall and the total height of the building is 64 ft. The four shop towers divide the interior of the facility into three bays. Each shop is a three-story steel braced frame with concrete floors and roof.

Vertical loads are carried by the steel truss and arch system, which is supported by the columns of the three shop towers. Spread footings beneath the tower columns support the vertical load at the foundation. The 22-ft deep truss acts as a horizontal diaphragm to transfer lateral load to the steel braced frames of the towers.

A rapid seismic evaluation consisted of calculating the shear capacity of the braced frames in the longitudinal and transverse directions. Additionally, the diagonal bracing was checked against buckling. The overturning capacity of the frame columns was calculated.

Building 2310 - Warehouse Supply

Building 2310 is a one-story building, approximately two stories tall. The building is basically rectangular in plan. The vertical load carrying system of the warehouse supply building consists of steel purlins spanning to prefabricated steel plate girder transverse frames simply connected at the midspan of the building. Column loads are supported by spread footings and CMU wall loads are supported by continuous strip footings. The lateral load resisting system is provided by a metal deck roof diaphragm and 12-in. reinforced CMU walls in the transverse direction, and steel rod braced frames in the longitudinal direction. The building appeared to be in good condition when inspected.

In the lateral analysis, researchers evaluated the capacity of the CMU walls and rod bracing of the steel frames. These braces are capable of resisting only tension.

Building 2418 - Officers' Dormitory

The officers' dormitory is a two-story, rectangular building. The maximum building height is approximately 18 ft. The vertical load carrying system consists of reinforced concrete slabs that carry loads to interior and exterior bearing walls. The bearing walls carry the load to continuous strip footings. The lateral load resisting system consists of reinforced concrete roof and floor diaphragms and CMU shear

walls in the transverse and longitudinal directions. The building appeared to be in good condition during the field inspection.

In the lateral analysis, researchers evaluated the capacity of the interior and exterior CMU walls. The exterior walls are penetrated by many openings.

Building 2595 - Child Care Center

The child care center is a single-story building with an addition. The maximum building height is approximately 15 ft. The vertical load carrying system consists of open-web joists and wide-flange steel beams that carry roof loads to steel pipe and tube columns, and exterior and interior bearing composite concrete/masonry walls. Continuous reinforced concrete strip footings support the superimposed loads and self weight of the walls. Pad footings support the columns. Lateral loads are resisted by a metal roof diaphragm and composite concrete/masonry walls in both the longitudinal and transverse directions. No structural problems were identified during the field inspection.

In the lateral analysis, the composite walls were assumed to act as concrete walls 3-in. thick. The masonry faces were not considered as resisting additional load.

Building 2605 - 15th AF Combat Operations

The combat operations center has two elevated floors and a basement. The building is rectangular in plan with an overall building height of 32 ft above finished grade. The vertical load carrying system consists of concrete joists and beams that carry the roof and floor loads to reinforced bearing concrete walls and columns. Continuous strip footings support the loads transferred by the walls. Pad footings support the loads from the columns. The lateral load resisting system consists of reinforced concrete slab roof and floor diaphragms, and cast-in-place concrete walls and concrete frames. The building appeared to be in good condition when inspected.

In the lateral analysis, the building was modeled as a two-story building with lumped masses at the roof and second floors. The first floor is at grade and all loads are assumed to be transferred into the foundation walls at this level.

Building 2630 - 33rd Communications Group Headquarters

An addition was made to the northwest corner of the original portion of the building. It is approximately rectangular in plan. The building is one-story with two roof levels; the height of the lower roof is approximately 16 ft and that of the higher roof is 20 ft. The vertical load carrying system of the building is a concrete roof slab supported by open-web steel joists and steel beams. Beams are supported by steel columns and precast concrete walls. All columns are supported by square footings. Continuous strip footings support the loads of all bearing walls. Lateral loads are transferred from the reinforced concrete diaphragm to the precast concrete shear walls of the original structure and the CMU shear walls of the addition.

In the lateral analysis of the building, an 18-ft average height was assumed. The weight of penthouses on top of the low roof were lumped at the roof level. The grout bed between the precast concrete walls and the foundation was assumed to govern the strength capacity of the lateral force resisting system in the original structure.

Building 2706 - NCO Club

The NCO club is a single-story building with a relatively small second story penthouse located at the center of the building. The building is extremely irregular in plan with many setbacks. The maximum building height at the roof is 14 ft. The building is constructed of a number of material systems.

The vertical load carrying system consists of open-web steel joists, steel beams, and tapered girders that carry the roof load to the concrete walls, or of concrete columns. The load is then transferred through the walls or columns to the footings. Continuous reinforced concrete strip footings support the loads of the bearing walls. Spread footings support the column loads. The lateral load resisting system consists of reinforced gypsum concrete, gypsum formboard roof diaphragm, and CMU shear walls in both the longitudinal and transverse directions. The building was in good condition when inspected.

In the lateral analysis of the building the concrete columns were not assumed to resist lateral loads. The interior and exterior CMU walls were assumed to resist the full seismic load.

Building 3403 - 15th AF Headquarters

The 15th AF headquarters is a two-story building with three airhandling penthouses above the roof level. The height to the top of the roof is 25 ft. It is T-shaped in plan; the cross wing of the T is asymmetrical in plan.

The vertical load carrying system consists of concrete slabs, joists, and beams placed integrally to carry the vertical loads to reinforced concrete walls and columns. Continuous strip footings support the load of the bearing walls; spread footings support the load from the columns. The lateral load resisting system consists of reinforced concrete roof and floor diaphragms, cast-in-place concrete walls, and reinforced concrete frames in both the longitudinal and transverse directions. No condition problems were identified during the field inspection.

In the lateral analysis of the building, the weight of the airhandling penthouses was lumped at the roof. Only centerline columns forming frames in the east-west oriented wing were assumed to resist load in that direction and only those forming frames in the north-south oriented wing were assumed to resist load in that direction.

Building 3407 - Dormitory

The airmen's dormitory is a standard design adapted to many AFB sites; similar buildings have been identified at Castle AFB. It is rectangular in plan with three stories and a partial basement. Each story is approximately 10 ft in height with an overall building height of 30 ft above grade. The vertical load carrying system consists of reinforced concrete roof and floors that distribute loads to reinforced concrete beams. These span to reinforced concrete columns supported by spread footings. The lateral forces are transferred by concrete floor and roof diaphragms to reinforced concrete frames. Frames are located along each perimeter face and along two lines in the interior of the building in the longitudinal direction. In the transverse direction, researchers assumed that the CMU infill walls resist the load as they are stiffer than the concrete frames. The building appeared to be in good condition during the inspection.

Building 3417 - Dormitory

This airmen's dormitory is rectangular in plan and three stories tall. Each story is approximately 10 ft tall; the overall height of the building is 30 ft. The vertical load carrying system consists of reinforced concrete flat slab roof and floors that transfer the loads to concrete columns. Spread footings support the column loads. The lateral forces are transferred by concrete floor diaphragms to concrete or CMU shear walls.

In the lateral analysis, the shear capacity in the longitudinal direction was assumed to be provided by the concrete and CMU shear walls. In the transverse direction, shear resistance is provided by concrete shear walls alone. While the columns may form a frame with the flat slab, such an analysis requires more detail than that included in the scope of this project. Therefore, researchers did not consider the columns as resisting seismic load.

Table 17 summarizes the buildings analyzed at March AFB.

Table 17

Description of Buildings Analyzed at March AFB

Bldg No.	Year Built	No. of Stories	Total Area	Type	Lateral Force Resisting System
300	1929	1	31,000	Steel/RC	RC shear walls
465	1932	1	17,000	Steel/ Clay MU	Clay MU
651	1956	2	24,000	CMU	CMU shear walls
760	1966	1	10,000	GSU	GSU shear walls
960	1972	1	85,000 CMU	Steel/ braced frame	CMU, Steel
962	1958	1	15,500	CMU	CMU shear walls
1220	1957	2		CMU	CMU shear walls
1220 (Twr)	1957	13			
1223	1956	2	14,000	CMU	RC/CMU shear walls
1305	1958	1	9,000	CMU	CMU shear walls
2300	1954	1	30,000	CMU/RC	RC frame w/ CMU shear wall infill
2303	1955	1	104017	Steel	Steel braced frame
2310	1967	1	8,000	Steel	CMU shear wall/ Steel braced frame
2418	1967	2	11,000	CMU	CMU shear wall
2595	1968	1	11,000	CMU/Steel	RC/CMU shear wall
2605	1963	3	19,000	RC	RC shear wall
2630	1954	1	61,000	Precast C/Steel	Precast C/CMU shear wall
2706	1956	1	21,000	RC/CMU	CMU shear wall
3403	1953	2	67,000	RC	RC shear wall
3407	1953	3	8,000	RC	RC frame, CMU shear wall
3417	1957	3	8,400	RC	RC/CMU shear wall

4 PRELIMINARY ANALYSIS

Building Properties

For the principal direction of each building, the following structural properties were calculated: base shear capacities at yield and ultimate based on the capacity of the primary lateral force resisting system; spectral accelerations at yield and ultimate; and natural periods of vibration at yield and ultimate. Physical properties were obtained from existing available design drawings. When information was not available, assumptions were made. Connections between horizontal diaphragms and vertical resisting elements in the lateral load resisting system were evaluated qualitatively to determine if they had the capacity to transfer load. Other assumptions and qualitative evaluations were made in calculating these properties in order to quickly assess the seismic vulnerability of each building.

Base Shear Capacities

Base shear capacities for the longitudinal and transverse directions of the buildings were computed at yield and ultimate levels. These quantities depend on the structural/material system of the building as well as its particular structural configuration. A primary part of these systems is the vertical element that resists lateral forces. These may be categorized generally as shear walls, braced frames, and moment frames. They all transmit lateral forces from the horizontal diaphragm above to the diaphragm below or to the foundation. The calculation of the base shear capacity of each vertical system that resists lateral forces is explained below.

Shear Walls. A wall that resists a horizontal force parallel to it is classified as a shear wall. The forces in these walls are predominantly shear forces, hence the name. Very slender walls may also resist forces in bending. In general, shear wall system capacities were based on the shear capacity of the composite materials which was modified by an adjustment for the flexural capacity that is dependent on the height-to-depth ratio of the wall piers.

Concrete Shear Walls. Two types of concrete shear walls are used in military construction: cast-in-place concrete shear walls and precast concrete shear walls. These two systems may behave quite differently under dynamic loading based on the vertical and horizontal continuity of the systems within a structure. The capacity of cast-in-place concrete shear walls depends on the quality of the reinforcement detailing. Their failure mode in earthquakes is typically due to overstress in shear and is generally ductile in behavior.

The major structural properties to be considered in calculating the base shear capacity of a building with a primary lateral force resisting system consisting of cast-in-place concrete shear walls is the shear stress, v, of the concrete and the reinforcing steel in the walls. Capacity is a function of the concrete strength, f'_c.

At ultimate:

$$v_u = (2 * \sqrt{f'_c}) + p_n * f_y \qquad \text{[Eq 1]}$$
$$V_u = A_c * V_u$$

where	V_u	=	shear force
	v_u	=	shear stress
	f'_c	=	compressive strength of the concrete (psi)
	p_n	=	ratio of distributed shear reinforcement on a plane perpendicular to plan of A_c calculated as: $A_S = A * h/[(h) * (hw)]$
	f_y	=	yield strength of reinforcement (psi)
	A_c	=	net area of concrete section bounded by web thickness and length of section in direction under consideration (in.).

For walls having a ratio h_w/l_w less than 2.0, the shear strength of the wall is calculated using the following equation:

$$v_u = (\text{alpha} * \sqrt{f'_c}) + p_n * f_y \qquad \text{[Eq 2]}$$
$$V_u = A_c * v_u$$

where alpha varies linearly from 3.0 for $(h_w/l_w) = 1.5$, to 2.0 for $(h_w/l_w) = 2.0$.

At yield:

$$V_y = V_u/1.5 \qquad \text{[Eq 3]}$$

where V_y = shear force.

The load factor, 1.5, closely approximates that used in the American Concrete Institute's (ACI) ultimate strength design procedure. Using this load factor assumes that the overall yielding of the lateral force resisting system of the building occurs at 2/3 of ultimate base shear capacity which, experience indicates, approximates the relationship between damage and strength for reinforced concrete.

The major difference between cast-in-place and precast concrete walls is the amount, type, and location of the vertical continuity reinforcing and the detailing of the roof and floor diaphragms to the walls. Often the roof and floor elements penetrate into the walls. There is an inherent weakness in unit construction versus monolithic construction under dynamic loading. Historically, many precast concrete shear wall buildings have not performed well in earthquakes, failing after exhibiting little ductility. Observed behavior in earthquakes shows that the system capacity is controlled by the elastic capacity of the precast panel elements and the inelastic capacity of their connections. The connections are particularly critical (cracking of joints is frequently exhibited after an earthquake) and therefore they must have sufficient strength to perform as monolithic construction.

Most of the precast concrete shear wall buildings analyzed can be categorized as single panel, coupled wall systems. In this system, there are no horizontal joints between precast panel elements and adjacent panels are connected. The control tower at March AFB is a large panel, coupled wall system; the precast panels are stacked vertically and adjacent panels are connected. The computed base shear capacities of precast concrete shear wall buildings were based on either the effective shear capacity of the panels themselves, when adequate connections were developed, or, in most cases, the shear capacity of the connection.

The shear capacity of the walls was determined similarly as for cast-in-place concrete shear walls. As stated above, connections are traditionally weak links in the seismic resistance of precast buildings. Wall-to-foundation connections are usually of four types: (1) thin grout joints, (2) grouted joints with

mechanical connectors, (3) welded joints, and (4) bolted joints. Each joint type should be analyzed in detail to ensure the structural integrity of the buildings is maintained under seismic loads.

The strength of thin grout joints in horizontal shear depends on the shear strength of the grout, friction and shear-friction, and the mechanical anchorages. Each of these capacities must be evaluated and since a crack along the horizontal plane may result, the lesser of the values should govern the overall capacity of the joint.

In joints without mechanical anchorages, shear resistance is provided by the friction between the two concrete surfaces. While section 6.8.a. of Technical Manual (TM) 5-809-10 states "the contact joint itself is a cold joint and will be given no shear or tension value" researchers assumed the capacity of horizontal joints without reinforcing was limited by the dead load friction associated with the superimposed and self weight of the walls. Frictional capacity depends on the Coulomb friction factor which characterizes the roughness of the two surfaces. For concrete placed against hardened concrete not intentionally roughened, the factor is 0.6. Shear strength is determined by:

At ultimate:

$$V_u = u * P_u \qquad \text{[Eq 4]}$$

where u = 0.6 lamda
lamda = 1.0 for normal weight concrete
= .85 for sanded lightweight concrete
= .75 for all lightweight concrete
Pu = the superimposed and self weight tributary to the wall.

This value must be less than or equal to:

$$V_u = v_{cg} * t_g * l \qquad \text{[Eq 5]}$$

where v_{cg} = grout shear strength <= .2 * f'_{cg} (psi)
where f'_{cg} = compressive strength of joint material <= 800 psi
t_g = joint width (in.)
l = joint length (in.).

At yield:

$$V_y = V_u/1.5 \qquad \text{[Eq 6]}$$

The Prestressed Concrete Institute (PCI) Manual[8] recommends a conservative value of 80 pounds per square inch (psi) shear-transfer strength for nonreinforced shear keys of diaphragm connections. A similar limiting value should be assumed here.

In joints with reinforcing, the connection capacity is based on shear friction. The effect of the normal force is provided by the reinforcing crossing the shear plane. The capacity depends on the roughness of the concrete surfaces, the percentage of reinforcing crossing the joint and the yield strength

[8] *Prestressed Concrete Institute Manual on Design of Connections for Precast Prestressed Concrete* (Prestressed Concrete Institute, 1973).

of the reinforcing. When reinforcement is present in the joint in shear action, tension is developed in the reinforcing producing compression in the concrete. The shear strength of this joint may be expressed as:

At ultimate:

$$V_u = A_{vf} * f_y * u \qquad \text{[Eq 7]}$$

where A_{vf} = area of shear reinforcement perpendicular to the shear plane (sq in.).

The PCI Manual recommends a maximum shear stress, v_u, of 420 psi for horizontal joints with reinforcement perpendicular to the shear plane.

At yield:

$$V_y = V_u/1.5 \qquad \text{[Eq 8]}$$

In this type of connection, significant shear displacements at the joint may be expected under large lateral loads. When the joint is opened, due to any significant uplift force, the shear friction capacity is lost; resistance to overturning is the gravity load transferred by the wall panel.

Masonry Shear Walls

Masonry is the predominant construction material in military construction. It is a composite consisting of units, mortar, and/or reinforcement and grout. Masonry systems are field assembled, and at least one primary component, the mortar, is field mixed without an accurate means of measuring the materials. The result is a significant lack of quality control and quality assurance. For this reason, its yield and ultimate capacities are difficult to assess.

Masonry construction may be unreinforced, reinforced and partially grouted at the locations of the vertical and horizontal reinforcement, or reinforced and solidly grouted (meaning all cells, those with reinforcement and without reinforcement, are grouted). Walls may also be of single or multiple wythes. For walls with reinforcing, the reinforcement ratios are checked relative to the minimum requirements in TM 5-809-10, Table 8-5. Because of the uncertainty of reinforcing in the existing masonry construction, all walls were analyzed as unreinforced walls and the deficiencies were noted in the analysis summaries.

Capacity of the shear wall is then calculated assuming that the weakest shear plane will yield at a nominal shear stress for the material. In evaluating masonry structures, researchers assumed that the shear strength of a wall is governed by the following equation:

At yield:

$$V_y = A_n * v_y \qquad \text{[Eq 9]}$$

where A_n = (face shell area) + (grouted core area) + (area of webs adjacent to grouted cores) (sq. in.)
v_y = 17 psi.

For fully grouted walls and for solid unit masonry, the net bedded area is equal to the gross area of the wall. The nominal yield strength for all masonry is considered to be v_y = 17 psi. This is the maximum allowable shear stress on masonry shear walls with a M/Vd > 1.0 (where M is the maximum moment

applied to the wall by lateral shear force and d is the length of the wall) and no special inspection in accordance with the 1985 Uniform Building Code (UBC) and discussed in Schneider and Dickey.[9]

At ultimate:

$$V_u = 1.5 * V_y \qquad \text{[Eq 10]}$$

Reinforced brick walls were analyzed somewhat differently. Reinforced brick walls are multi-wythe brick walls with a reinforced concrete or grout collar joint typically 3 to 4 in. thick. Two methods can be used to analyze the capacity of these systems: (1) the wall can be analyzed as a masonry wall, taking the net area as the gross area of the entire wall, or (2) lateral load may be assumed to be resisted solely by the collar joint, assuming it acts as a reinforced concrete shear wall. Researchers assumed that the higher value of the two methods constituted the capacity of these shear walls.

Masonry veneer carries no vertical load and is connected to the primary structural system using wall ties. These systems provide no lateral force resistance to the structure. Masonry veneers actually pose an additional life safety hazard. Degradation of wall ties or inadequate spacing can result in loss of the veneer in out of plane ground motions.

Braced Frames

Braced frames are similar to shear walls in their general function and stiffness. The elements of the frames are primarily subjected to axial forces. The capacity of a braced frame depends on the tensile or compressive capacity of the brace, or the capacity of the connections of the braced frame members to one another and the balance of the building. The principal types of braced frames are the concentrically braced frame (CBF), eccentrically braced frame (EBF) and knee-braced frame (KBF). Eccentrically braced frames have been used only recently in buildings and will not be found in structures constructed before 1983. For this reason, they will not be discussed further here. While braced frames may be constructed of wood, concrete, or steel, the majority are steel. The rest of this section will therefore refer to braced frames constructed of steel.

Most buildings with braced frame vertical elements in their lateral force resisting system are CBFs. Several configurations of CBFs exist; some of these exhibit poor response under dynamic loading. The CBF configurations are defined below.

Diagonal or X-Bracing

One or a pair of diagonal braces cross from a beam-column joint at the top of a column to a beam-column joint at the base of an opposite column to form this bracing configuration. In x-bracing, the pair of braces will cross at their midlength. A single brace will resist axial forces in either tension or compression depending on the direction of the force. In a pair of braces, one will resist forces in tension and the other in compression, if it is capable (Figure 4).

[9]Schneider, Robert R., and Walter L. Dickey, *Reinforced Masonry Design*, Second Edition, (Prentice-Hall, Inc., 1987) pp 277-278, Table A-3.

Chevron or V-Bracing

In this type of CBF, a pair of braces is located either above or below a beam. They terminate at a single point, usually the centerline, within the beam clear span. When braces intersect the beam from above, the configuration is called V-bracing; from below it is called chevron bracing (Figure 4). If one brace should fail, the axial force in the remaining brace cannot be resisted except by the beam. This additional force may adversely affect beam behavior and alter the response of the braced frame system.

K-Bracing

A pair of braces is located on one side of a column and terminate at a single point within the column clear height in this brace configuration (Figure 4). Failure in one brace of the system will cause the other brace to exert a lateral force on the column. This could adversely affect the column behavior possibly causing column buckling and catastrophic failure at this story level. Therefore, this configuration is no longer allowed in the seismic design of new buildings. It should be evaluated in detail in a vulnerability assessment.

Knee-Braced Frame

The beam and column are usually joined with a simple connection; a brace spans from a point along the height of the column to a point along the length of the beam (Figure 4). When the knee brace is relatively short, most of the frame deflection is due to flexure in the beams and the columns, and the frame should be treated as a moment frame. However, when the knee brace is relatively long, most of the frame deflection is due to axial deformation in the members and the frame should be treated as a braced frame.

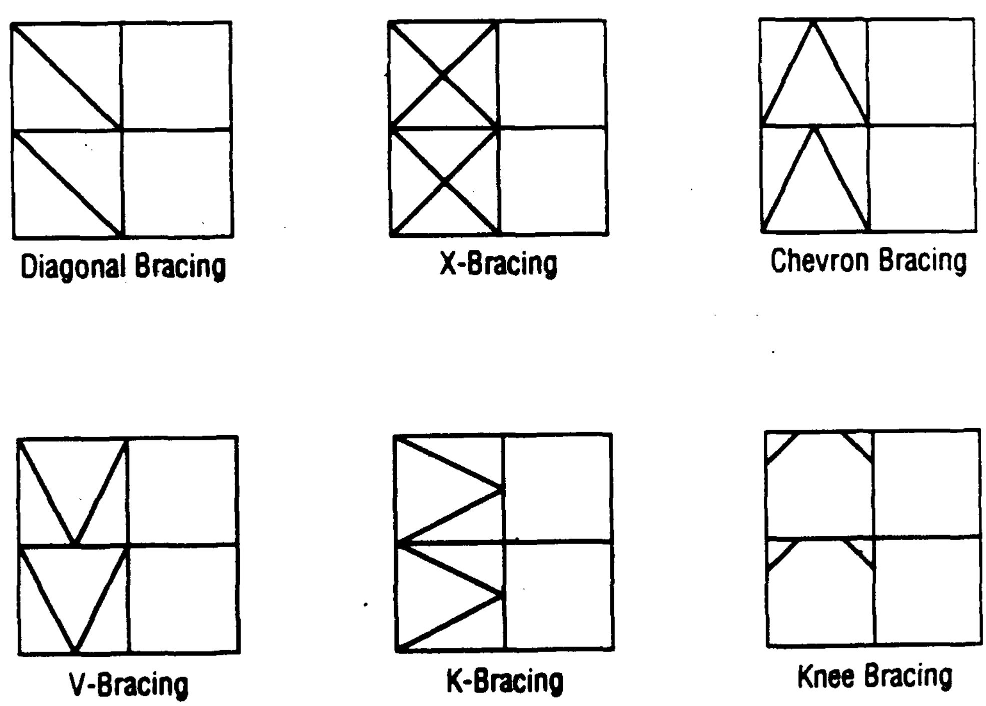

Figure 4. Concentrically braced frames.

The capacity of braced frame systems is controlled by the weaker of the brace capacity in tension or compression and the brace connection with the beams and columns. Additionally, the connection between the columns and the foundation should be checked to ensure that the shear capacity of the braces and connections can be developed here as well. The ultimate tension capacity in the brace is computed using the following equation:

$$V_{ubrace} = A * f_u * \cos theta \qquad \text{[Eq 11]}$$

where A = area of the brace (sq in.)
f_u = yield stress (ksi)
(for A36 steel this was assumed to be 36 ksi)
theta = the angle the brace forms with the horizontal plane.

Yield stress was similarly calculated substituting f_y of 30 ksi for f_u.

Compressive capacity depends on slenderness of the member. A very slender brace has essentially no compressive capacity. Researchers assumed that braces with a slenderness ratio greater than $720/(f_y)^{1/2}$ (120 for f_u = 36 ksi) are effective in tension only. The slenderness ratio is expressed as:

$$L/r \qquad \text{[Eq 12]}$$

where L = the unsupported length of the brace (in.)
r = the radius of gyration (in.).

$$V_{ubrace} = A * f_a * \cos theta \qquad \text{[Eq 13]}$$

where f_a = compressive stress, (ksi) calculated as
([12 * pi^2 * E]/[23 * (L/r)2])/
(1.6- [L/(200 * r)]) where E = modulus of elasticity of steel.

To evaluate the strength of riveted or bolted connections of diagonal bracing the following equations are used:

At yield:

$$V_y = N_r * A_v * f_y * \cos theta \qquad \text{[Eq 14]}$$

where N_r = number of rivets or bolts for connections in single shear
Av = cross-sectional area of each rivet or bolt (sq in.).

At ultimate:

$$V_u = V_y \qquad \text{[Eq 15]}$$

Moment Frames

Moment resistant frames provide seismic resistance by bending and shearing of columns and beams, connected by moment connections. Because this structural system is more flexible than a braced frame or shear wall system, the drift of the frame under lateral load is an important characteristic affecting the

overall behavior of the building and its contents. It is particularly important to evaluate the deflection associated with response of the frame in its inelastic range.

Concrete Frames

Reinforced concrete frame capacity is based on a relationship between the flexural capacity of the columns and their shear strength. If a column's capacity in shear is less than the shear associated with the flexural capacity of the column, a brittle failure of the column can occur resulting in possible collapse. Older columns with ties spaced equal to the depth of the column are particularly vulnerable. Additionally, older frame systems where the beams are shallow or are formed by the column strips of flat slabs usually do not meet detail requirements for ductile behavior. Detailing that characterizes ductile behavior includes

1. Beam stirrups, column ties, and joint ties which ensure that the shear capacity of the members exceeds the shear associated with flexural capacity.

2. Confinement of concrete in locations where plastic hinge formation will form.

3. Adequate longitudinal reinforcement at the bottom of beams to provide positive moment capacity at the beam ends where plastic hinges will form.

4. Long lap splices located out of regions of high moment confined by transverse reinforcement.

5. Members proportioned to form strong column/weak beam systems.

To evaluate the shear capacity of the frame columns, the capacities due to both direct shear and that shear associated with the moment capacity should be evaluated. The shear capacity of the member should be greater than the resulting shear from the moment capacity or brittle failure may result. If the moment capacity of the element is the limiting condition, a more ductile failure will result. This requires that the element not be overreinforced (i.e., the flexural steel yields before crushing of the concrete in the compressive zone). The shear capacity due to direct shear is:

$$V_u = [2 * (f'_c)^{1/2} * b_w * d] + [(A_v * d)/s * f_y] \qquad \text{[Eq 16]}$$

where V_u = average shear stress in a representative column (psi)
b_w = web width (in.)
d = distance from extreme compression fiber to centroid of longitudinal tension reinforcement, (in.)
A_v = area of shear reinforcement within spacing, s (sq in.)
s = spacing of shear reinforcement in direction parallel to longitudinal reinforcement (in.)
f_y = yield strength of reinforcement.

The shear related to the moment capacity of the element is:

$$V_u = 2 * M_u/L \qquad \text{[Eq 17]}$$

where L = length of the element (in.)
M_u = moment capacity of the member (in.- kips [k]) tension bars are assumed to be at $1.25 * f_y$, capacity should account for axial load in element.

Building Shear Capacity

The total shear capacity of the building is a sum of the shear capacity of all of the elements resisting load at each of the two levels: yield and ultimate. For buildings with a lateral load resisting system of shear walls, this is the sum of the shear capacity of each wall element. The weakest horizontal plane of the wall is found by excluding the area of door and window openings. Wall piers with a height-to-length ratio greater then four were also excluded. These piers will be significantly more flexible than other walls, resisting load predominately by flexure versus shear behavior, and will carry significantly less horizontal load. Similarly, for lateral load resisting systems that are solely of braced frames or moment resisting frames, the total capacity of the building is the sum of all frames.

Calculating the capacity of systems with more than one type of lateral load resisting element is more complex. Due to the variation in stiffness of the different systems and the way in which they are combined, they may not act simultaneously. For example, capacity analysis of buildings with reinforced concrete frames and infill shear walls is based on two primary assumptions: (1) concrete frames are nonductile and therefore have very little lateral force resisting capacity and (2) the masonry is significantly stiffer than the concrete frames. Researchers assumed the frames would not effectively carry any load until the masonry has failed. The capacity of this system was therefore based on the shear strength of the infill walls only. The capacity of the frames is not considered.

In buildings with combined braced frame and shear wall lateral load resisting systems, researchers assumed that the load was resisted simultaneously by both elements. Braced frames and shear walls have more comparable relative stiffnesses than do shear walls and moment frames.

Base shear coefficients, C_{by} and C_{bu}, at yield and ultimate were then calculated by dividing the total building base shear, V, by the total building weight, W:

$$C_b = V/W \qquad \text{[Eq 18]}$$

The base shear coefficient is then converted to spectral accelerations at yield and ultimate, S_{ay} and S_{au}, by dividing C_b by a constant representing the effective modal weight at the story of interest. The effective modal weight factor is computed using the equation:

$$\text{alpha} = (\text{Sum m} * \text{phi})^2/(\text{sum m})(\text{sum m} * \text{phi}^2) \qquad \text{[Eq 19]}$$

where m = the mass (weight/gravity [w/g]) at each story
phi = the mode shape at each story.

For one-story buildings, this factor is 1.0. The spectral acceleration therefore reflects the influence on base shear, V, of story mass distribution and modal participation.

$$S_a = C_b/\text{alpha} \qquad \text{[Eq 20]}$$

Natural Periods

Computed natural periods at yield were based on the empirical formula:

$$T_y = 0.05\ h_n/\ [(D)^{1/2}] \qquad \text{[Eq 21]}$$

where T_y = natural period at yield (sec)
h_n = height of building (ft)
D = base width of building for the direction under consideration (ft).

The natural period at ultimate is related to the yield period and the calculated spectral accelerations. It was computed using the formula:

$$T_u = T_y \, [u \, (S_{ay} / S_{au})]^{1/2} \qquad \text{[Eq 22]}$$

where T_u = natural period at ultimate (sec)
u = ductility factor
S_{ay} = spectral acceleration at yield (g)
S_{au} = spectral acceleration at ultimate (g).

Damping and Ductility Values

Table 18 gives the assumed damping values, *B*, used in the analyses.

The ductility, *u*, approximates the energy dissipation that occurs during inelastic material behavior. The values assumed are given in Table 19 and are related to the ratio of the ultimate deflection to the yield deflection.

Capacity Spectra

In this rapid evaluation, researchers approximated the capacity of the building to resist lateral load and presented this as a curve formed by points representing initial major yielding and ultimate strength. These points were based on the natural period at yield, T_y, and the building shear capacity at yield, V_y; and the ultimate natural period, T_u, and the ultimate building shear capacity, V_u. This capacity curve was plotted against the demand spectra modified by the damping values characteristic of each material system. An example of such a curve is shown in Figure 5.

Table 18

Assumed Damping Values

Building Type	Percentage of Critical Damping	
	Yield	Ultimate
Steel	3	7
Concrete	5	10
Masonry	5	10

Table 19

Ductility Factors

Building Type	*u*
Steel	4
Concrete	3
Masonry	2

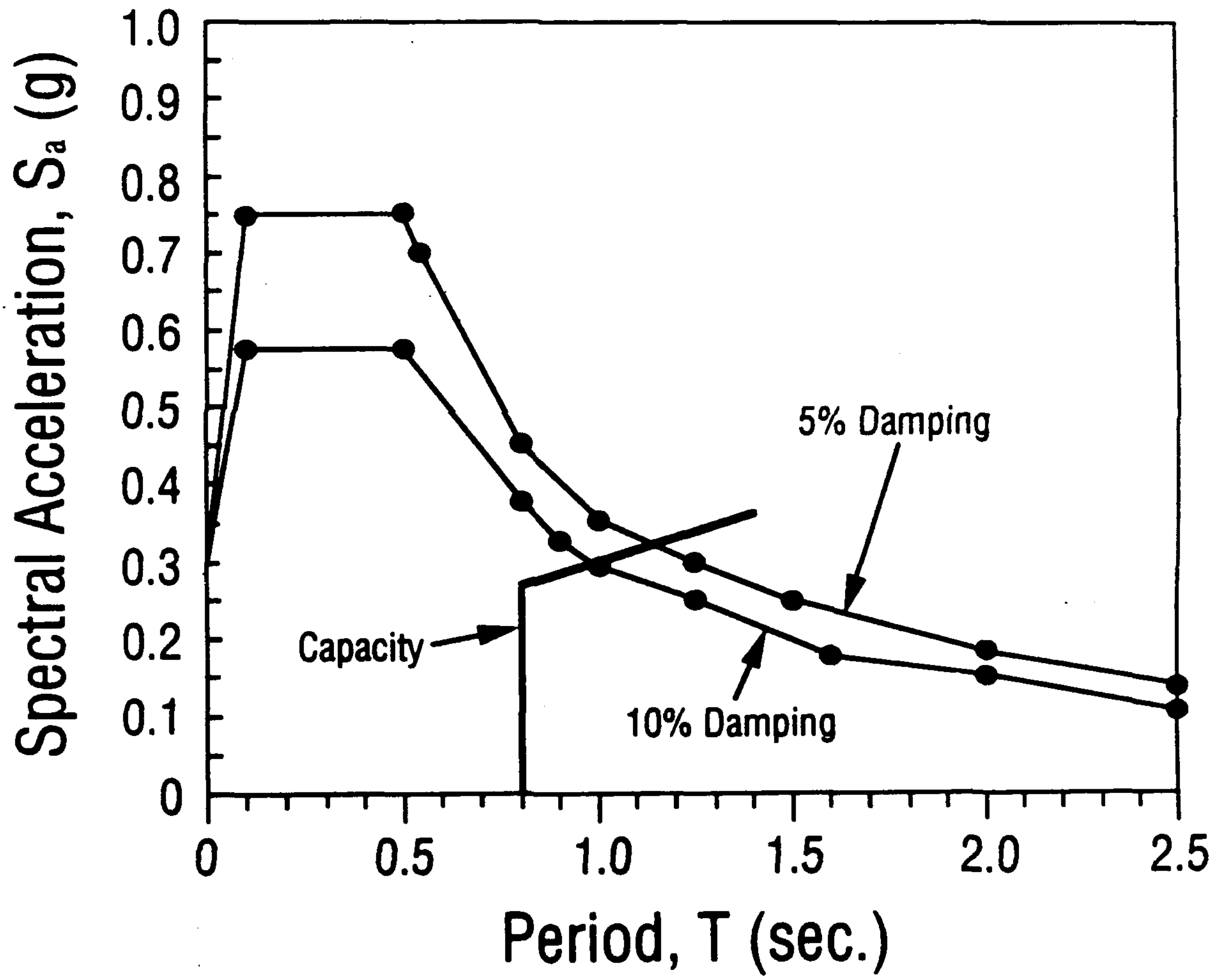

Figure 5. Capacity curve vs demand.

5 ANALYSIS

Preliminary Analysis Results

The results of the capacity evaluation are expressed in terms of the fundamental period of vibration and spectral acceleration. Two capacities were calculated for each of the principal directions of the building; one represents initial major yielding and the other represents the ultimate strength of the lateral force resisting system.

Tables 20, 22, and 24 summarize the computed natural periods for the buildings analyzed at Beale, Castle, and March AFBs, respectively. Spectral accelerations at yield and ultimate levels are summarized in Tables 21, 23, and 25.

Table 20

Natural Periods (in Seconds) of Buildings Analyzed at Beale AFB

Building	Yield		Ultimate	
No.	Long.	Trans.	Long.	Trans.
1025	0.069	0.081	0.097	0.115
1029	0.020	0.029	0.028	0.041
1060 (main)	0.027	0.043	0.038	0.061
1060 (tower)	1.12	1.20	1.72	1.84
1086 (main)	0.045	0.041	0.064	0.057
1086 (A-VI)	0.073	0.044	0.10	0.061
1200	0.067	0.078	0.076	0.090
2145 (main)	0.161	0.215	0.227	0.303
2145 (power)	0.102	0.161	0.144	0.227
2340	0.068	0.069	0.078	0.080
2418	0.084	0.098	0.097	0.114
2434	0.049	0.069	0.070	0.098
2459	0.098	0.099	0.160	0.161
2474	0.091	0.147	0.105	0.170
2490	0.048	0.056	0.055	0.065
5700 (main)	0.053	0.042	0.075	0.059
5700 (nursing)	0.027	0.037	0.038	0.052
5700 (clinic)	0.021	0.045	0.030	0.064
5800	0.042	0.049	0.049	0.057

Table 21

Summary of Spectral Accelerations (in Grams) at Beale AFB

Building No.	Yield Long.	Yield Trans.	Ultimate Long.	Ultimate Trans.
1025	0.04	0.04	0.06	0.06
1029	0.29	0.19	0.44	0.29
1060 (base)	1.59	0.77	2.39	1.16
1060 (tower)	0.48	0.48	0.81	0.81
1086 (main)	0.32	0.25	0.48	0.38
1086 (A-VI)	0.13	0.46	0.19	0.73
1200	0.44	0.18	0.67	0.27
2145 (main)	0.47	0.44	0.71	0.66
2145 (power)	1.08	0.76	1.56	1.14
2340	0.10	0.18	0.15	0.27
2418	0.40	0.25	0.60	0.37
2434	0.17	0.14	0.25	0.22
2459	0.13	0.13	0.20	0.20
2474	0.77	0.93	1.39	1.16
2490	0.35	0.21	0.52	0.31
5700 (main)	1.08	0.90	1.63	1.35
5700 (nursing)	2.10	1.30	3.15	2.00
5700 (clinic)	*	*	*	*
5800	0.57	0.58	0.85	0.86

*Review of the structure shows only the first floor requires upgrading.

Table 22

Natural Periods (in Seconds) of Buildings Analyzed at Castle AFB

Building No.	Yield Long.	Yield Trans.	Ultimate Long.	Ultimate Trans.
175	0.144	0.173	0.204	0.245
360	0.064	0.070	0.090	0.099
752	0.050	0.059	0.058	0.069
759	0.041	0.043	0.048	0.050
786	0.139	0.172	0.197	0.243
1212	0.101	0.246	0.141	0.284
1230	0.047	0.108	0.054	0.125
1260	0.109	0.126	0.154	0.178
1325	0.057	0.087	0.066	0.151
1335	0.050	0.051	0.058	0.059
1340	1.040	1.110	1.450	1.570
1344	0.072	0.093	0.083	0.107
1350	0.064	0.100	0.098	0.153
1360	0.071	0.092	0.100	0.130
1532	0.055	0.064	0.064	0.074
1540	0.080	0.153	0.092	0.177
1582	0.067	0.078	0.076	0.090

Table 23

Summary of Spectral Accelerations in Grams at Castle AFB

Building No.	Yield Long.	Yield Trans.	Ultimate Long.	Ultimate Trans.
175	0.655	0.655	0.982	0.982
360	1.270	0.553	1.910	0.830
752	0.487	0.868	0.729	1.300
759	0.352	0.346	0.528	0.519
786	0.479	0.214	0.719	0.321
1212	0.150	0.022	0.226	0.033
1230	0.489	0.301	0.733	0.453
1260	0.812	0.779	1.220	1.170
1325	0.465	0.182	0.670	0.272
1335	0.292	0.263	0.438	0.395
1340	1.120	0.992	1.680	1.490
1344	0.858	0.928	1.290	1.390
1350	1.010	2.480	1.730	4.220
1360	0.553	1.370	0.830	2.060
1532	0.416	0.574	0.624	0.862
1540	0.423	0.335	0.633	0.505
1582	0.444	0.182	0.670	0.273

Table 24

Natural Periods (in Seconds) of Buildings Analyzed at March AFB

Building No.	Yield Long.	Yield Trans.	Ultimate Long.	Ultimate Trans.
300	0.04	0.04	0.05	N/A
465	0.09	0.10	0.10	0.12
651	0.07	0.07	0.08	0.08
760	0.08	0.13	0.09	0.15
960A	0.07	0.07	0.08	0.08
960B	0.07	0.11	0.08	0.12
960W	0.12	0.13	0.18	0.19
962	0.04	0.05	0.05	0.06
1220	0.08	0.11	0.11	0.15
1220T	0.74	0.66	1.26	1.12
1223	0.09	0.09	0.11	0.11
1305	0.07	0.08	0.08	0.09
2300	0.04	0.04	0.04	0.04
2303	0.06	0.10	0.10	0.15
2310	0.06	0.09	0.10	0.10
2418	0.05	0.11	0.06	0.13
2595	0.03	0.03	0.05	0.05
2605	0.05	0.08	0.07	0.11
2630	0.03	0.03	0.04	0.04
2706	0.04	0.05	0.04	0.05
3403	0.03	0.04	0.05	0.06
3407	0.10	0.14	0.25	0.28
3417	0.05	0.12	0.08	0.17

Table 25

Summary of Spectral Accelerations (in Grams) at March AFB

Building No.	Yield Long.	Yield Trans.	Ultimate Long.	Ultimate Trans.
300	1.38	N/A	2.06	N/A
465	0.03	0.07	0.04	0.10
651	0.47	0.29	0.70	0.43
760	0.48	0.29	0.72	0.44
960A	0.17	0.20	0.26	0.30
960B	0.37	0.20	0.55	0.31
960W	0.13	0.08	0.23	0.14
962	0.19	0.32	0.28	0.48
1220	0.36	0.16	0.55	0.25
1220T	1.42	1.53	1.88	2.05
1223	0.53	1.07	0.80	1.35
1305	0.44	0.18	0.67	0.27
2300	0.25	0.32	0.37	0.47
2303	1.01	2.48	1.73	4.22
2310	0.59	0.81	0.89	1.21
2418	0.20	0.10	0.30	0.16
2595	0.53	1.40	0.79	2.09
2605	1.30	0.81	1.93	1.22
2630	0.23	0.20	0.34	0.31
2706	0.30	0.33	0.44	0.50
3403	0.76	0.58	1.14	0.76
3407	0.19	0.03	0.28	0.04
3417	0.17	0.52	0.25	0.79

Capacity vs Demand

Building capacity was reconciled to earthquake demand, and damage was estimated if capacity did not meet demand. The objective is to estimate the damage ratio due to EQ-II. For each building, the natural periods at yield and ultimate levels versus the associated capacities in terms of spectral acceleration are plotted on the demand spectra. The percent damage is resolved graphically between the demand and capacity curves. At yield, damage is assumed to equal zero and damping was assumed to be constant up to the yield limit. At ultimate, damage is assumed to be 100 percent and damping is the greater value as defined for each material in Table 18. In this analysis, damping and damage are assumed to vary linearly between the yield capacity, S_{ay}, and the ultimate capacity, S_{au}. An example of this procedure is shown in Figure 6. Combined damage for the building as a whole is computed based on the sum of 2/3 of the damage in the critical direction and 1/3 damage in the other direction. Damage for each of the buildings analyzed is summarized in Tables 26, 27, and 28.

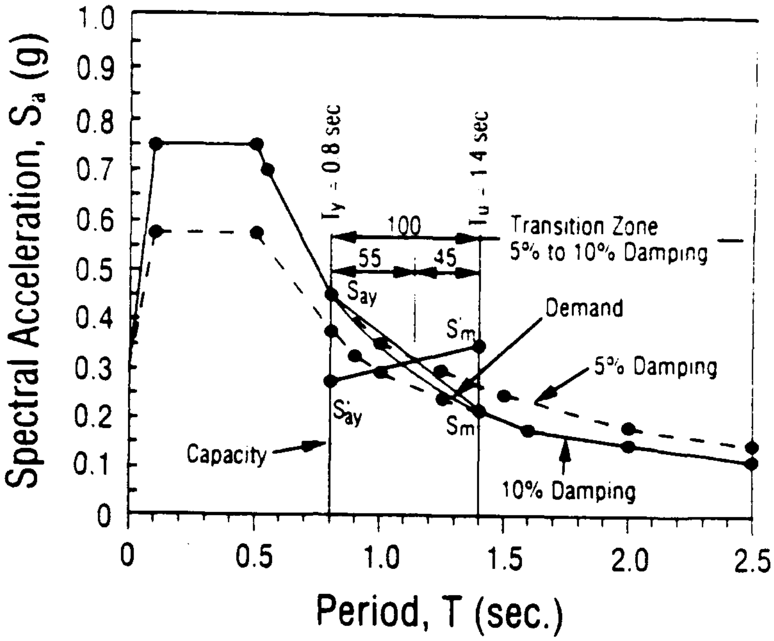

Figure 6. Percentage damage.

Table 26

Building Damage and Upgrading Cost at Beale AFB

Building No.	Estimated Damage for EQ-II Long. (%)	Trans. (%)	Comb. (%)	Area (sf)	Upgrade Cost ($/sf)	Total Cost ($K)
1025	100	100	100	78000	90.48	7057
1029	100	100	100	16444	111.11	1827
1060(BO)	0	0	0			
1060(CT)	10	4	8			
1086(main)	100	100	100	172408	82.25	14181
1086(A-VI)	68	100	89	14300	82.25	1047
1200	67	100	89	8424	82.97	622
2145(main)	71	81	78	41556*	97.69	3167
2145(Pwr)	0	0	0			
2340	100	100	100	7838	144.30	1131
2418	92	100	97	23300	98.70	2231
2434	100	100	100	42100	64.36	2709
2459	100	100	100	22150	64.36	1426
2474	0	0	0			
2490	100	100	100	15970	144.30	2304
5700	0	0	0			
5800	45	43	44			
					TOTAL	37,702

* Only the first floor of this structure requires upgrading.

Table 27

Building Damage and Upgrading Cost at Castle AFB

Building No.	Estimated Damage for EQ-II Long. (%)	Trans. (%)	Comb. (%)	Area (sf)	Upgrade Cost ($/sf)	Total Cost ($K)
175	5	4	5			
360	0	30	20			
752	51	0	34			
759	100	100	100	51433	64.36	3310
786	52	100	84	8395	117.17	826
1212 (6)*	100	100	100	24638	66.52	9834
1230	57	100	86	28396	66.52	1625
1260	0	0	0			
1325	62	100	87	5376	49.21	230
1335	100	100	100	29568	66.52	1967
1340	0	0	0			
1344	0	0	0			
1350	0	0	0			
1360	45	0	30			
1532	78	20	59	13338	43.80	345
1540	75	100	92	11395	66.02	692
1582	67	100	89	8424	66.52	499
					TOTAL	$19328

*Total number of buildings is given in parentheses.

Table 28

Building Damage and Upgrading Cost at March AFB

Building No.	Estimated Damage for EQ-II Long. (%)	Trans. (%)	Comb. (%)	Area (sf)	Upgrade Cost ($/sf)	Total Cost ($K)
300 (8)*	0	100	67	31000	57.72	9600
465	100	100	100	16560	98.70	1634
651	100	100	100	24000	144.30	3470
760	100	100	100	10354	117.17	1213
960(A)	100	100	100	46000	64.36	2961
960(B)	100	100	100	9600	64.36	618
960(W)	100	100	100	26550	64.36	1709
962	100	100	100	15446	144.30	2228
1220	100	100	100	9414	113.42	1058
1220(Twr)	0	0	0			
1223	100	45	82	15638	87.30	1140
1223(Twr)	100	100	100			
1305	100	100	100	8892	86.38	770
2300	100	100	100	30000	66.02	1980
2303	0	0	0			
2310	100	42	81	80000	43.80	2838
2418 (3)	100	100	100	10944	86.58	2843
2595	100	0	67	11300	97.69	732
2605	7	69	48			
2630	100	100	100	60728	123.81	7451
2706	100	100	100	21336	144.30	3079
3403	79	93	88	67400	113.42	7109
3407 (2)	100	100	100	24585	86.58	4258
3417 (2)	100	100	100	25232	86.58	4370
				TOTAL	61,061K	

*Total number of buildings is given in parentheses.

Upgrading Costs

A Federal Emergency Management Agency (FEMA) publication[10] states there "...is a lack of information on the true costs of seismic rehabilitation." While relatively little data is available to estimate the cost of upgrading facilities to improve their seismic resistance, researchers have attempted, in a very simplistic manner, to compute these costs to arrive at an approximate idea of the scope of needed work. Although the information may be used to prioritize facility upgrades at a base, it should not be considered an accurate construction cost.

Facilities that were calculated to have 60 percent damage or greater resulting from EQ-II were reviewed. Upgrade costs (see Tables 26, 27, and 28) were computed based on the percent damage, total square footage of the facility, and the cost of new construction for the building function as prescribed in Army Regulation (AR) 415-17.[11] Cost factors were adjusted to account for 3 percent inflation per year since 1980 and for the regional construction costs in California. Indirect costs due to mission interruption, and occupant relocation are not estimated here. More detailed cost estimates require a more detailed assessment of each building's structural capacity and deficiencies. Such analyses were not included in this project.

Upgrading Concepts

Based on the preliminary analysis performed on the facilities at Beale, Castle, and March AFBs, general upgrading concepts are suggested. Detailed analysis will identify the specific vulnerabilities of each building and allow for the development of detailed upgrading concepts. A more accurate construction cost estimate may then be derived from this information. This was not within the scope of this work. Following are general considerations inherent to upgrading all structural systems and techniques relevant to each of the lateral force resisting systems existing at the bases.

Selection of the strengthening technique is based on the existing structural system, degree of deficiency, and several other general considerations. The global consequences of removing, adding, or modifying any structural or nonstructural element on the structural system must be adequately evaluated. This includes assessing the impact of the strengthening scheme on the plan configuration of the building. An irregular plan may be modified to reduce its effects in several ways; one method is to isolate portions of the building with highly dissimilar seismic response.

Any modifications to the eccentricity between center of mass and center of rigidity should also be considered when locating new elements, and strengthened or eliminating existing elements. The deformation compatibility of new and existing elements must be evaluated to ensure that new rigid elements, or those strengthened and stiffened, are more rigid than existing elements with lesser capacity. The new or modified structural elements should be designed to carry a significant portion of the lateral load thereby reducing the load within the capacity limits of the existing weaker elements.

The connecting elements of the structure must also be strengthened. Horizontal diaphragms and the connections tying the lateral system together may require upgrading to transfer the load demand to the strengthened elements. Additionally, the foundation system may require modification.

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[10] *Typical Costs for Seismic Rehabilitation of Existing Buildings* (Federal Emergency Management Agency, 1988).

[11] Army Regulation (AR) 415-17, *Construction Cost Estimating for Military Programming* (Headquarters, Department of the Army, February 1980).

Other factors including the need to maintain the operation of the facility, the opportunity to relocate the function, or additional alterations to the building based on other requirements will influence selection of the upgrading concepts. These factors must be considered in the development of any detailed upgrading concept; however, they were not considered in the development of general concepts for upgrading specific lateral force resisting systems.

Shear Walls

When limited additional capacity is required of existing buildings with lateral force resisting systems of unreinforced or lightly reinforced masonry shear walls, reinforced concrete may be added to the exterior or interior face of the existing walls. Additionally, in reinforced concrete and masonry shear wall buildings, wall openings may be filled with reinforced concrete to increase the shear area of the wall. If the capacity of the building must be significantly increased, replacement of existing unreinforced walls with reinforced concrete walls is recommended. New shear walls may also be added to buildings with reinforced walls. These walls are typically constructed of reinforced concrete, but may be steel. As discussed above, the placement of these new walls within the structural plan is critical to ensuring that eccentricity between the center of mass and center of rigidity is minimal. Additionally, shear walls should be located above one another in elevation so forces may be directly transferred from one floor to another. The new walls must be positively connected to the roof and floor diaphragms to ensure force transfer. Exterior steel or concrete buttresses may also be used to increase the lateral capacity of the building with a primary lateral force resisting system of shear walls.

Steel Braced Frames

Several options are available to reduce the vulnerability of a building with a steel braced frame lateral force resisting system. Any upgrading of the existing frames should strive to improve the compressive capacity as well as the tensile capacity of the brace members. In braces with some compressive capacity, this is most commonly done by reducing the slenderness (l/r) ratio of the brace by increasing the effective area of the member. Existing diagonal braces may also be removed and replaced with larger members in existing frames. New braced frames may also be added to the structural system to increase the overall capacity of the lateral force resisting system. Because of their deformation compatibility, new exterior or interior reinforced shear walls may be added to an existing braced frame building.

Reinforced Concrete Moment Frames

This lateral force resisting system is more complex to upgrade. The existing system is extremely difficult to modify to improve its behavior. Typically, upgrading of a building with this structural system requires adding new lateral force resisting elements. Internal or external steel frames with adequate rigidity may be used or, more commonly, new interior or exterior reinforced concrete shear walls are added. These may infill existing frame bays. Steel or concrete exterior buttresses are also a means of improving the seismic response of a reinforced concrete moment frame building without altering the existing frame elements significantly.

6 CONCLUSIONS AND RECOMMENDATIONS

Concrete Frame Construction

The seismic vulnerabilities of buildings at Beale, Castle, and March AFBs are summarized below. Research and lessons learned from earthquakes have identified the severe vulnerability of several structural systems commonly used in public construction. Many of these systems are prevalent in the building inventory of military installations. Of particular concern are older reinforced concrete frames, unreinforced masonry, and precast concrete systems. The vulnerabilities of each of these systems are discussed below.

In moment-resisting concrete frames, detailing of the reinforcement is the overriding parameter governing seismic performance. Adequate confinement of the concrete, defined by sufficient ties or spiral reinforcing in the columns and stirrups in the beams is critical to providing the ductility required to prevent collapse of the structure. A lack of confinement, exhibited in most of the concrete frame construction before the mid-1970's, may lead to brittle failure and structural collapse. Almost all of the buildings at Beale, Castle, and March AFBs having lateral force resisting systems incorporating reinforced concrete frames were constructed before the mid-1970's. The buildings greater than one story in height were found to have damage in excess of 60 percent; most exhibited 100 percent damage, due to the earthquakes of the magnitude used in the evaluation (EQ I and EQ II). They include Building 1212 at Castle AFB and Buildings 2300 and 3407 at March AFB. Reinforced concrete frame buildings that exhibited limited damage from the preliminary analysis, but should nonetheless be investigated further due to the brittle nature of the structural system, are Building 2145 at Beale AFB. and Buildings 1260 and 1344 at Castle AFB. Because the failure of these buildings under dynamic loading can occur without sufficient warning to ensure safety, it is recommended that detailed analysis and upgrading of all buildings with reinforced concrete frame lateral force resisting systems constructed before 1971 be given the highest priority in reducing the seismic vulnerability at the bases.

Strengthening of reinforced concrete frames is extremely complex because of the difficulty in providing confinement and shear reinforcement in beams, columns, and beam-column panel zones where a ductility is required. Typically, new structural elements are added to resist the majority of the lateral load thereby reducing the demand on the existing frames. When evaluating the cost of upgrading the building, consideration must also be given to moving the building function to another facility or constructing a new facility. This option may prove to be more economical than modifying the existing structure.

Precast Concrete Construction

The capacity of precast concrete shear wall buildings constructed of individual panels and connected by welded plates or reinforcing bars is highly dependent on the integrity of the connections between the panels and the foundation. Precast concrete structural elements must be tied together well to perform adequately under dynamic loading. Reinforcement must also be provided at the edges of members to resist load reversals and overturning forces.

Adjacent panels may be rigidly connected such that significant relative movement between the panels is not allowed. As an alternative, flexible connections may be provided between vertical elements that permit relative movement. Structures with rigid connectors have little ductility as they do not allow for inelastic deformation of the structure under high load levels; flexible connectors will slip, deform, or

yield and dissipate energy under high load levels. However, it is generally inappropriate to consider precast shear wall systems as ductile systems.

Masonry Units

Based on this preliminary analysis, many buildings with lateral force resisting systems of CMU or other masonry units including brick, gypsum, and clay tile exhibited a large degree of damage at the three bases. Buildings with large expanses of shear walls usually perform well in earthquakes; however, where shear walls are penetrated by many openings, severe shear cracking can occur in the remaining spandrel beams or wall piers tying the wall segments together. Boundary elements in shear walls are also important to ensure effective behavior due to flexural stresses in the walls. Reinforcement detailing that ensures good behavior in both flexure and shear in masonry walls was not typically provided in the older structures. While they may not suffer catastrophic collapse as is characteristic of nonductile reinforced concrete frames, the building may be irreparably damaged in a major seismic event.

Reinforcement

Many of the precast concrete buildings on the three bases resist lateral load by shear friction at the joints of the individual precast elements. While this is an acceptable method of resisting gravity loads, the Prestressed Concrete Institute recommends that reinforcing be provided across the shear friction joint a ductile connection under load reversals. The only resistance to overturning in joints without reinforcing is the gravity load transferred by the wall panel. This may be overcome by overturning forces associated with lateral load and the joint may open up. Force reversal and the associated rocking motion will result in damage at the edges of the wall panels and instability of the overall panel.

This reinforcement is lacking in all of the buildings with precast concrete shear wall lateral force resisting systems at Beale, Castle, and March AFBs. Dry pack grout typically connects the walls to the foundations in most buildings. Beale AFB buildings with precast concrete shear wall systems are: 1025, 1029, and 1086. The later two buildings have precast concrete systems combined with reinforced concrete shear walls and steel braced frames. Building 786 at Castle AFB has a lateral force resisting system of precast concrete shear walls as does Building 2630 at March AFB. The lack of reinforcement in the critical joints of these panelled buildings makes them very susceptible to severe damage and collapse in a major seismic event. It is recommended that these buildings be analyzed in detail and that comprehensive upgrading concepts be developed to reduce their potential failure due to seismic activity.

Damage Assessment

At Beale AFB, the analysis of Buildings 1200, 2340, 2418, 2434, 2490, and 5800 showed major damage due to the demand earthquake. Damage greater than 60 percent was calculated for Buildings 759, 1230, 1325, 1335, 1532, 1540, and 1582 at Castle AFB and Buildings 465, 651, 962, 1220, 1305, 2418, and 2706 at March AFB. Additionally, several buildings with combined lateral force resisting systems of either masonry shear walls and reinforced concrete shear walls or steel braced frames also exhibited major damage, including Buildings 960, 2310, 1223, 2595, and 3417 at March AFB. The greater number of vulnerable buildings at March AFB relative to the other two bases is not due to a change in the quality of construction but to the higher demand of the earthquake loading at this site.

While the great majority of masonry shear wall buildings appear highly vulnerable, the results of this analysis must be considered in light of the current knowledge of the material response to seismic loading. Definitive information is not currently available on the ultimate capacity of unreinforced and lightly reinforced masonry. The low value assigned to the material in this analysis significantly contributes to the large damage estimates associated with buildings with this lateral force resisting system. Further study of the ultimate strength of the material would be cost effective and would help to determine a more accurate estimate of the material's capacity; the buildings may have greater capacity than estimated in this analysis.

Connections

Several limitations associated with this preliminary analysis should be highlighted and addressed in the detailed analyses of those buildings with severe damage potential. This analysis evaluated only the yield and ultimate capacities of the vertical elements of the primary lateral force resisting systems of the identified mission essential and high potential loss buildings at Beale, Castle, and March AFBs. While the connections between elements of the lateral force resisting systems were not evaluated in detail, they were reviewed in concept. The capacity of connections to transfer load and sustain deformation compatibility with the elements of the systems may be limited. This situation is highlighted in connections between rigid walls and flexible roof diaphragms as is characteristic of some masonry buildings and connections between precast concrete elements. It is essential that connections develop the strength and ductility of the individual elements. Critical connections are those between roof and floor diaphragm elements, diaphragm element to wall element, vertical elements to one another at vertical and horizontal discontinuities, and vertical element to foundation. It is recommended that future detailed analysis be conducted to identify the specific vulnerabilities of each building.

Cost Estimates

Despite the lack of data available to use in accurately estimating the cost of upgrading to improve the seismic resistance of facilities, approximations were developed. Although this information may be useful when prioritizing facility upgrades, a more detailed assessment of each building would be needed to develop accurate construction costs.

At Beale AFB, costs range from $64,36 to $144.30/sq ft. Costs at Castle AFB range from $49.21 to $117.17/sq ft, and costs at March AFB range from $43.80 to $144.30/sq ft.

METRIC CONVERSION TABLE

1 ft	=	0.304 m
1 in.	=	2.54 cm
1 psi	=	703.1 kg/m^2
1 sq in.	=	6.452 cm^2
1 sq ft	=	0.093 m^2
1 lb	=	2.2 kg

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